Design of the Eurimbla Way Bridge

Author

Angelo Maio, Bridge Engineer
Kin Yap, Team Leader, Bridges & Structures

Abstract

To support the Commonwealth's continuous shipbuilding program at the Osborne Naval Shipyard, a grade separated road was proposed to link Pelican Point Road to Mersey Road North over the existing rail corridor in Osborne, South Australia.

The Eurimbla Way Bridge is a 380m long, 10-span simply supported Super T structure supported on reinforced concrete headstocks, columns, blade walls and continuous flight augured (CFA) piles. The structure accommodates a two-lane dual carriageway and one shared use path and is fitted with protection screens along its length. Its alignment incorporates a large vertical and horizontal curve and is skewed over the rail corridor. Challenging ground conditions, tight site constraints, and proximity to live ARTC rail lines resulted in complex design and detailing considerations.

This paper outlines the key design considerations, bridge detailing, and engineering challenges addressed during delivery of the Eurimbla Way Bridge.

1.0 Introduction

Eurimbla Way Bridge (formerly referred to as the Link Road Bridge) is a grade-separated crossing that carries road traffic over the existing rail corridor, providing a strategic connection between the western and eastern sides of the Osborne precinct. The bridge forms a key enabling element for development of the north-eastern LeFevre Peninsula as an expansion of the Osborne Naval Shipyard. By removing conflicts with rail operations, the new link road provides uninterrupted traffic movement and materially improves safety and network efficiency within the precinct.

This paper presents the following aspects of the project and bridge design:

- Project & bridge overview
- Functional and physical constraints on site
- Geotechnical risk and durability assessment
- Design development of the bridge at different stages
- Eurimbla Way Bridge description
- Seismic liquefaction design Deflection wall removal methodology

2.0 Project & Bridge Overview

Eurimbla Way Bridge is located in Osborne, South Australia, approximately 20 km north-west of Adelaide (refer Figure 2-1). The structure forms critical enabling infrastructure for the Commonwealth's continuous shipbuilding program at the Osborne Naval Shipyard, providing a direct connection between Pelican Point Road and Mersey Road North. By eliminating delays associated with the existing level crossing, the bridge improves network efficiency and reduces congestion as the surrounding facilities continue to expand.

SMEC Australia (SMEC) was engaged by Australian Naval Infrastructure (ANI) to undertake the detailed design of the road connection and Eurimbla Way Bridge between Pelican Point Road and Mersey Road North. The project provides a grade-separated link over the existing Australian Rail Track Corporation (ARTC) and privately-operated rail corridor to the shipyard. The design was developed in accordance with the Department for Infrastructure and Transport (South Australia) Master Specifications, together with the relevant Austroads Guides to Design and applicable Australian Standards.

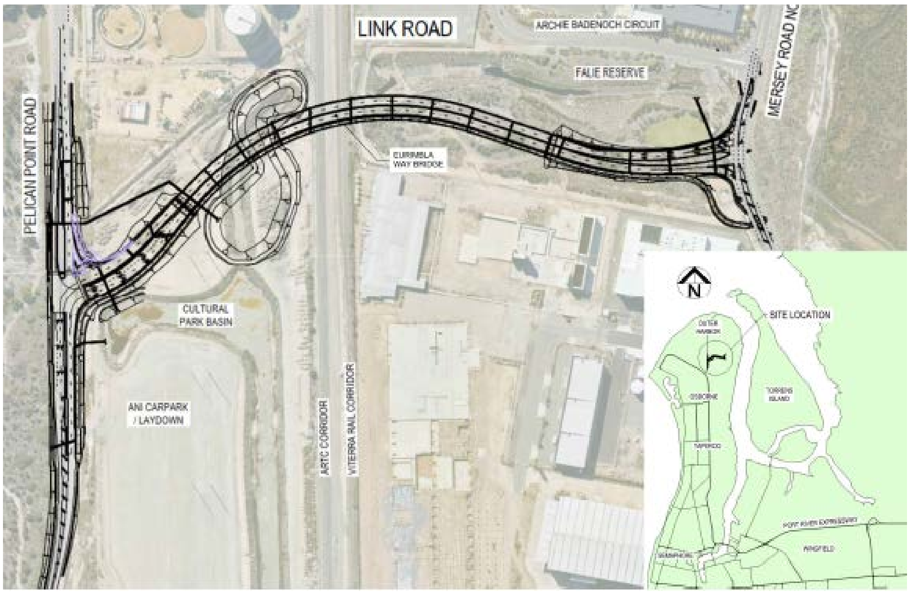

Figure 2-1: Locality Map of Eurimbla Way Bridge

3.0 Key functional and Physical constraints on site

The site is subject to a number of functional and physical constraints that influenced both the road geometry and bridge alignment. These constraints are summarised below:

- Detention basin, which limits the position and the span of the bridge
- The existing underground culvert over the rail corridor to the north of the bridge
- The SAPN overhead and underground services over the rail corridor to the south of the bridge
- The Ampol fuel terminal at the northwest side of the bridge.
- Osborne Naval Shipyard development at the southeast side of the bridge
- Rail Corridor, which limits the position of the piers and requires specific height clearances to the bridge soffit.

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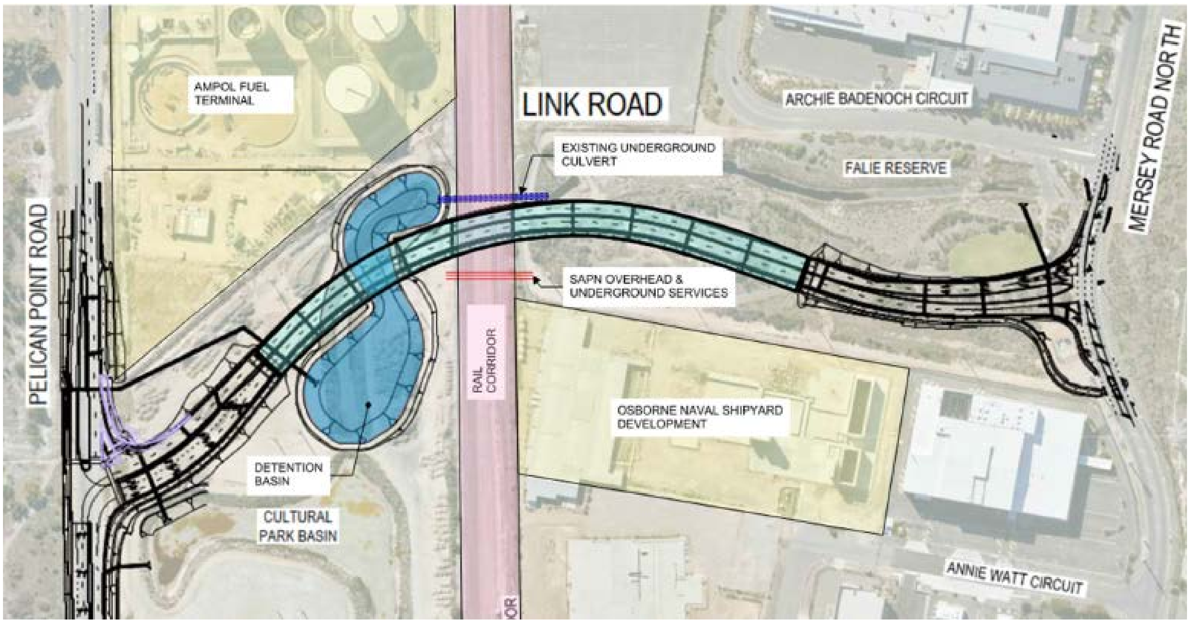

Figure 3-1: Site Constraints around Eurimbla Way Bridge

4.0 Geotechnical Risk

The bridge is located on the LeFevre Peninsula, where approximately 7.5 m of estuarine marine soils of the St Kilda Formation overlie the Hindmarsh Clay layer. Groundwater is shallow, having been encountered at approximately 2 to 3 m below existing ground level.

Settlement is one of the principal geotechnical risks associated with the project due to the underlying St Kilda Formation, which is predicted to induce significant primary and creep settlements where earth fill is placed, particularly at and behind the abutments and within the approach embankments. A further challenge is the variability and uncertainty of the formation's organic content and geotechnical characteristics. This inherent variability may manifest as localised soft and stiff zones in close proximity, with differential settlement becoming apparent only over time as creep progresses. Even under relatively low embankment heights, the untreated condition is expected to result in subsidence and subsequent pavement cracking.

Geotechnical modelling also indicates a risk of soil liquefaction during a major earthquake event (1 in 1,000 year return period or greater), which was required to be considered in the design of the bridge substructure.

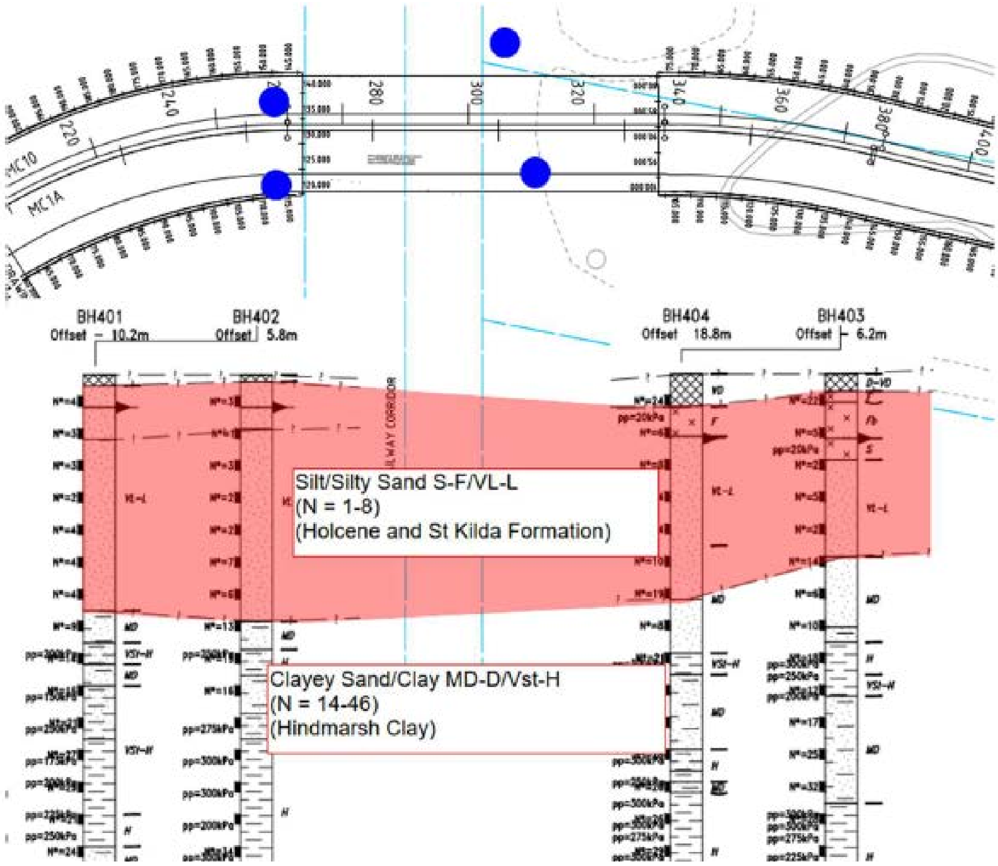

Figure 4-1: Simplified Geotechnical Longitudinal Sections

5.0 Durability Assessment

Laboratory testing indicated that soils in the vicinity of the bridge contain chloride concentrations of up to 19,000 mg/kg, magnesium concentrations of up to 17,000 mg/kg, sulphate concentrations of up to 2,500 mg/kg, and ammonia concentrations of up to 76 mg/kg. On the basis of this soil aggressivity, an AS 5100.5 exposure classification of C2/U was adopted, requiring the use of special concrete mixes. For CFA piles, concrete mix design was further informed by BRE Special Digest 1 (SD1) to address the elevated contaminant levels, particularly magnesium.

Above ground, the bridge was classified as exposure classification B2, reflecting its proximity to Gulf St Vincent (approximately 1 km) and the Port Adelaide River (approximately 600 m).

6.0 Design Development of the Bridge

6.1 Reference Design Stage

At the reference design stage, the bridge was designed as a two-span Super T structure, comprising a 37.5 m span over the rail corridor and an 18 m eastern span that provides for a north–south internal access road. Reinforced soil structure (RSS) walls were proposed to retain the surrounding fill at both abutments. An additional culvert was provided beneath the RSS wall on the western side to convey flows from the detention basin (refer Figure 6-1).

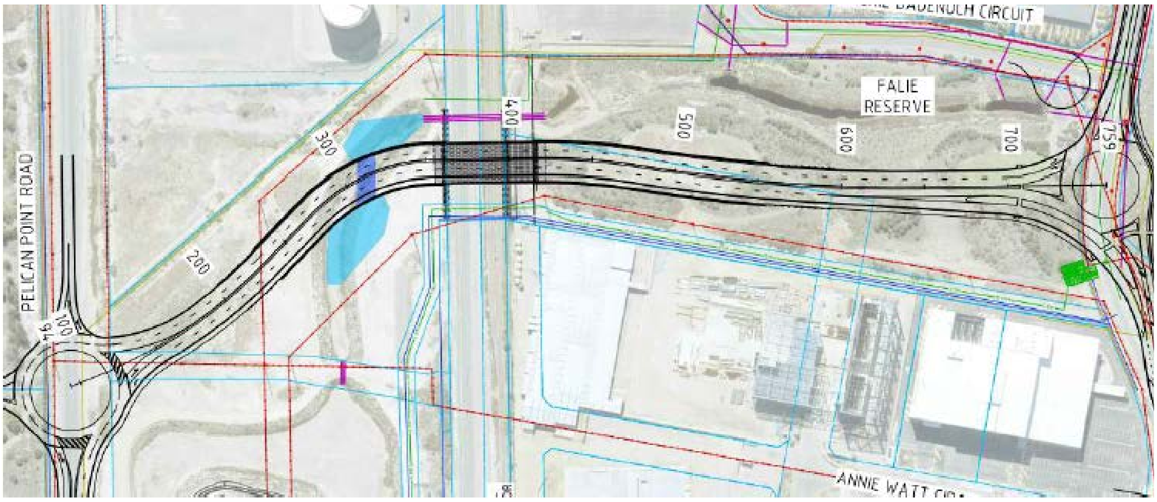

Figure 6-1: Bridge Geometry During Reference Design (Plan)

6.2 Functional Design (30%) Stage 1

As the design progressed to the 30% concept stage, several issues were identified with the reference design solution:

- RSS wall heights of up to 10 m at both abutments were predicted to result in significant settlements, exceeding 300 mm (primary) and 70 mm (creep)
- Potential differential movement between the culvert and the adjacent RSS wall within the basin area.

To meet the settlement criteria specified in DIT Master Specifications RD-EW-D1 (Earthworks for Roads) and ST-RE-D1 (Design of Reinforced Soil Structures), a bridge optioneering study was undertaken for ANI. The following options were developed to achieve compliance;

- Option 1 – Maximise the two bridge spans (up to 38 m), with both eastern and western embankments constructed using RSS walls supported by controlled modulus columns (CMC). Preloading would also be required at the abutments, with early estimated duration of 9-24 months due to the large fill heights and poor founding soils.

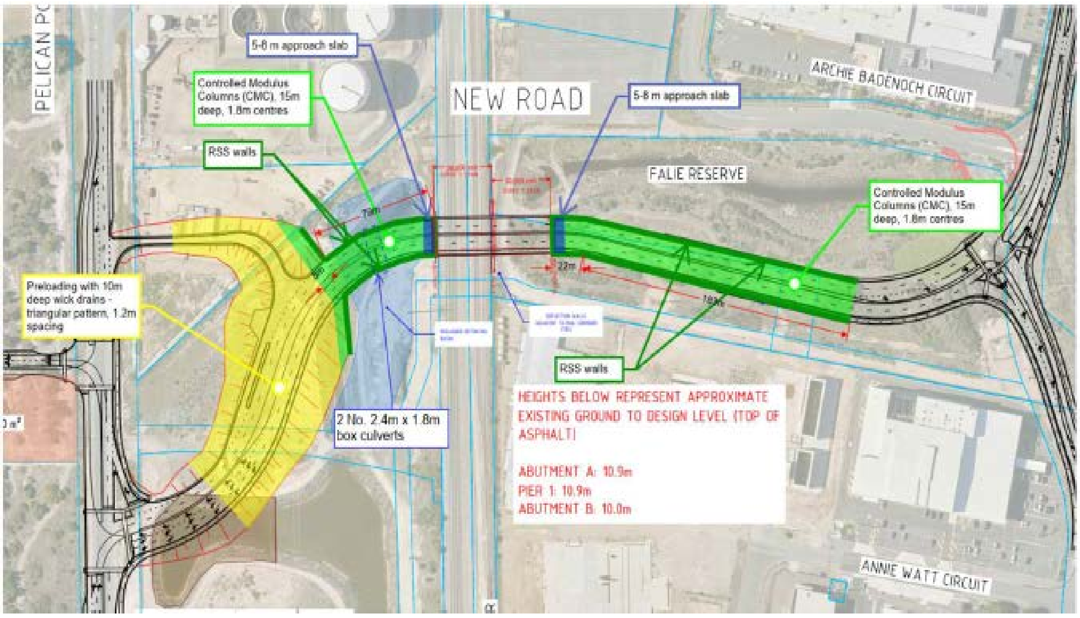

Figure 6-2:Optioneering Option 1

- Option 2 – As per Option 1, but adopting two 50 m steel girder spans (with a structural depth comparable to the proposed Super T options). This approach increases clearance at the western abutment over the rail corridor and allows for “spill-through” abutments at both the eastern and western ends. It also reduces the extent of required ground improvement due to the smaller embankment footprint.
- Option 3 – A multi-span Super T solution comprising a seven-span arrangement, with two additional spans to the west and three to the east. The geotechnical approach is consistent with Option 1; however, the extent of eastern earthworks is significantly reduced. The western culvert is removed, enabling the detention basin to potentially be expanded to increase storage capacity.

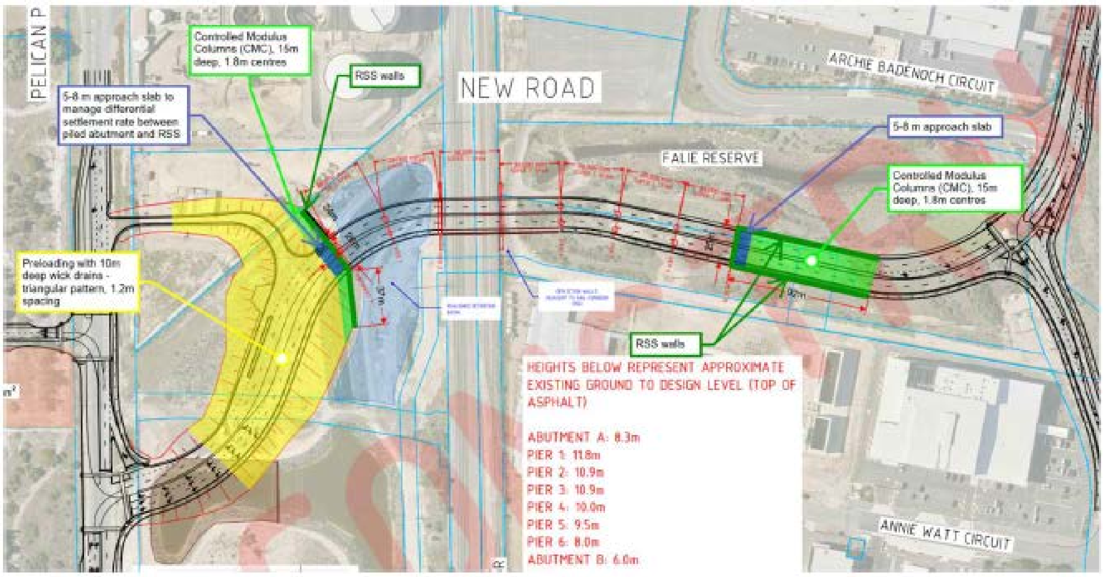

Figure 6-3:Optioneering Option 3

- Option 4 – Similar to Option 3, but incorporating a steel girder for the rail span to achieve increased clearance over the rail corridor.

Following design reviews and workshops, Option 3 comprising a seven-span Super T configuration was selected at this stage. This option reduced preloading requirements and enabled a shorter construction program, alongside minor optimisations to the road geometry (refer Figure 6-4).

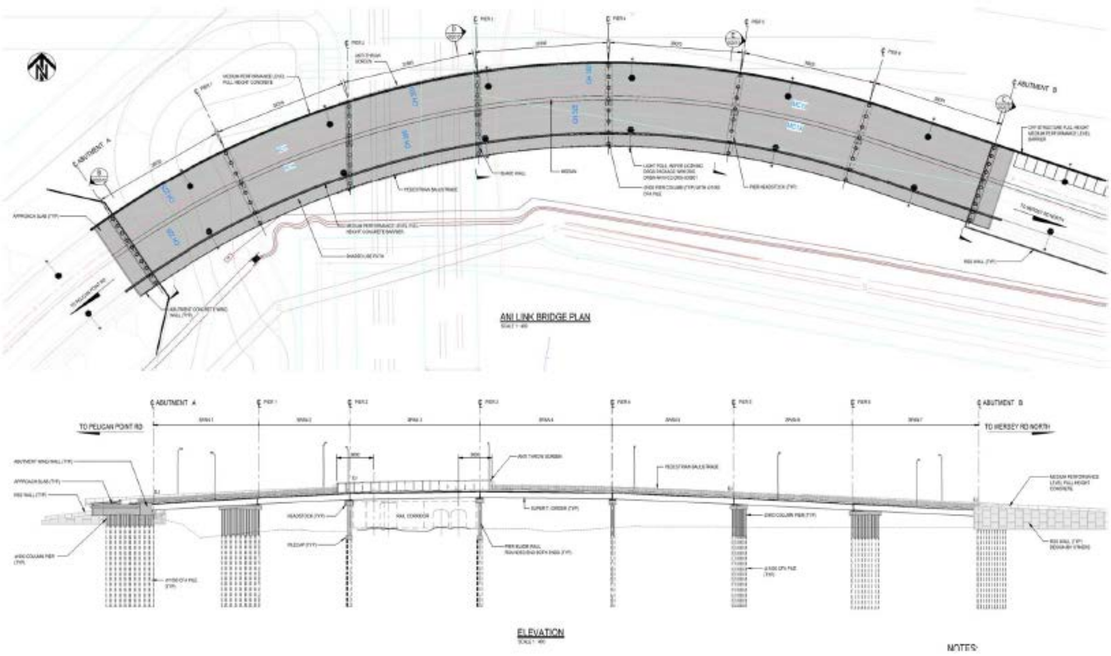

Figure 6-4: Concept Design (30%) Solution

6.3 Detailed Design (70%) Stage 2 & Full Design (100%) Stage 3

During the 70% Stage 2 design development, following Early Contractor Involvement (ECI) review, it was suggested that a 10-span Super T option be investigated due to the following advantages:

- Elimination of RSS walls at the abutments, thereby removing associated procurement risks and replacing them with batters and low-height retaining walls.
- Further reduction in earthworks, with fill heights limited to approximately 4 m. This allows compliance with DIT Master Specification settlement criteria using controlled modulus columns (CMC) and preloading, without requiring any design departures and while meeting the construction program.

To meet the design schedule, the bridge substructure design (piles and columns) was accelerated to facilitate early procurement of reinforcement cages. Refer Figure 6-5 for bridge layout.

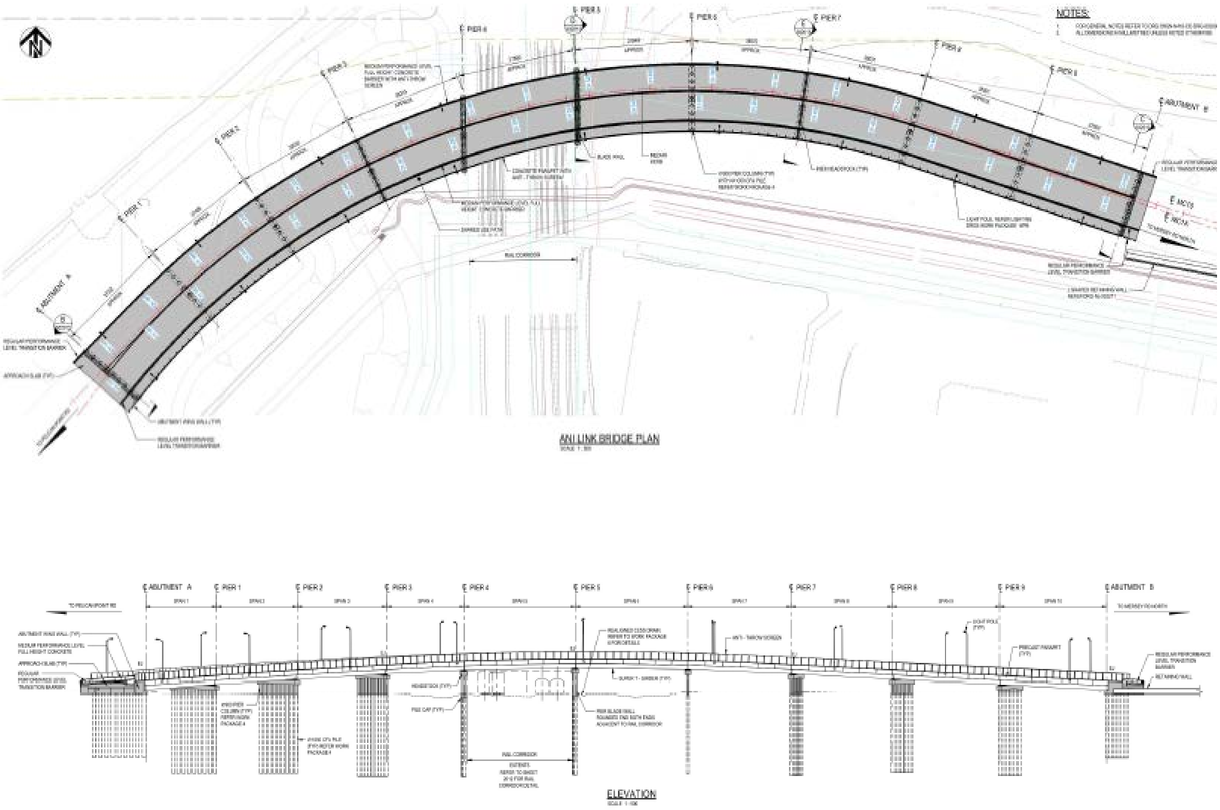

Figure 6-5: Detailed Design (70%) & Full Design (100%) Solution

6.4 Design Development Summary

In consultation with the client and construction team, a 10-span bridge configuration was ultimately selected to support an accelerated construction programme. Increasing the number of spans to meet the project timeline proved to be effective, with the adopted scheme enabling full project delivery in a shorter duration than the preloading periods anticipated in the reference design. The construction of the 380 m long, 10-span bridge was completed in approximately 12 months. Its final structural form is described in Section 7.

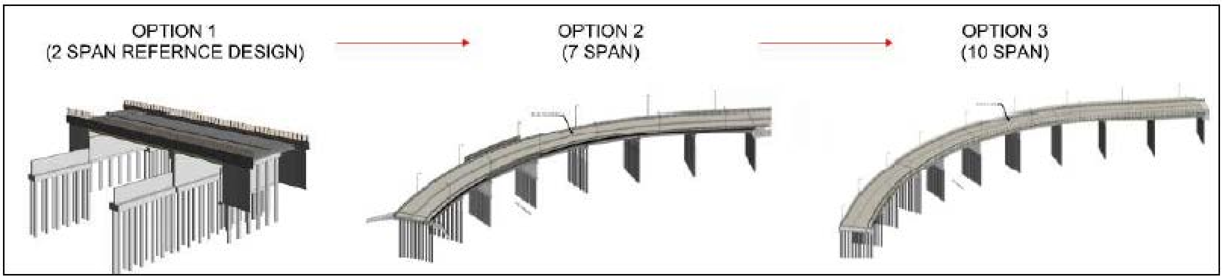

Figure 6-6: Design Development Figure

Table 6-1: High Level Option Comparison

Option	PROS	CONS
OPTION 1 (Ref. Des.) (2 Span)	• Less structure to construct • Simpler structure alignment and detailing (straight)	• Large fill heights at abutments requiring extensive preloading (approx. 9m fill) • 9–24 months of preloading required prior to construction to accelerate consolidation settlement and creep • RSS retaining walls required at abutments
OPTION 2 (7 Span)	• Lower fill heights behind • abutments compared to Option 1 • Reduced preloading period • compared to Option 1	• Slightly more complex structure alignment • Substantial preloading still required (several months) for abutments • RSS retaining walls required at abutments • Additional costs (in comparison to Option 1) associated with concrete elements, precast elements, and handling
OPTION 3 (10 Span)	• Very low fill heights at abutments • Minimal preloading required at abutments • No RSS retaining walls required at abutments • No idle time • Shorter construction program	• More structural elements to maintain in the future • More complex structure alignment • Additional costs (in comparison to Option 1) associated with concrete elements, precast elements, and handling

7.0 Eurimbla Way Bridge Description

7.1 General Description

The Eurimbla Way Bridge is a 10-span, simply supported structure with an overall width of 21.5 m. It is designed to accommodate two-way traffic, with two lanes in each direction, together with a shared use path (SUP). Span 3 crosses the realigned detention basin, Span 5 crosses the rail corridor, and Span 6 provides clearance for a future internal access road.

The bridge incorporates varying skew, with each span nominally 38 m in length, resulting in an overall length of approximately 380 m. A minimum vertical clearance of 7.1 m is provided over the rail corridor at Span 5 to satisfy ARTC Structure Gauge Outline F requirements, while a minimum clearance of 5.4 m is maintained at Span 6 for the future access road.

Figure 7-1: Revizto model of the bridge over the rail corridor

7.2 Bridge Superstructure

Each span comprises 11 Super T girders, each 1.815 m deep, acting compositely with a cast in-situ reinforced concrete deck slab that is at least 220 mm thick. The bridge deck is finished with an 85 mm thick asphalt wearing surface.

On the southern side, additional concrete fill is provided on the deck to achieve the required shared use path (SUP) design level. Safety screens, extending 3 m above any climbable surface or final deck level, are provided along both the northern and southern edges for the full length of the bridge.

7.3 Bridge Substructure

7.3.1 Abutments

Both abutments comprise reinforced concrete sill beams supported on nine continuous flight auger (CFA) piles, each 1,050 mm in diameter. Owing to the aggressive ground conditions at the site, a concrete cover of 120 mm was adopted for the CFA piles. The piles are approximately 21 m long at both Abutment A and Abutment B and are founded in Hindmarsh Clay.

The sill beams are designed to accommodate future bridge jacking, allowing bearing replacement if required. A 5 m long approach slab is provided at each end to facilitate a smooth transition between the roadway and the bridge deck. Batter slopes are formed at the front and sides of each abutment to support the retained ground.

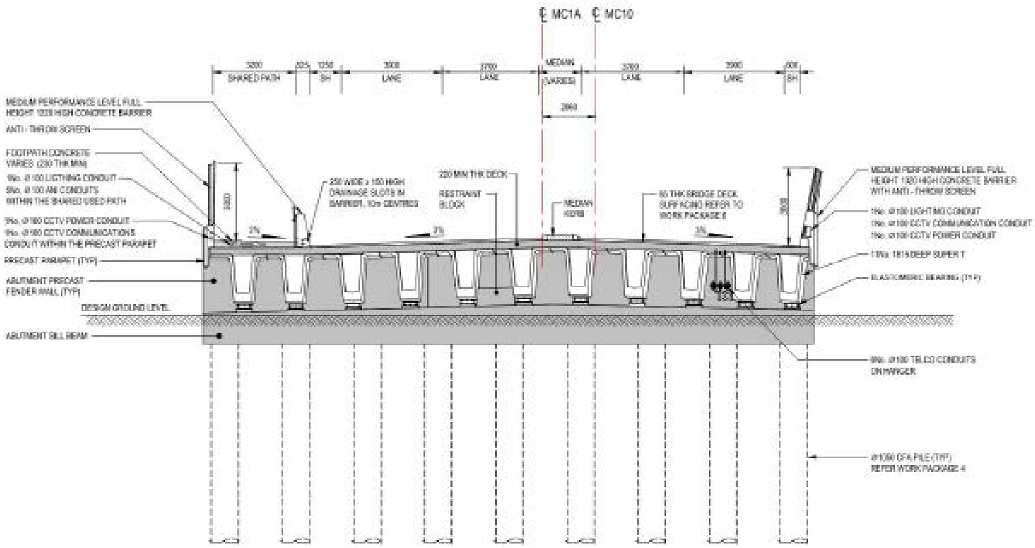

Figure 7-1: Typical Abutment Section

7.3.2 Piers

At typical column piers the superstructure is supported on headstocks founded on nine reinforced concrete column extensions, each 900 mm in diameter. These columns are supported by 1,050 mmdiameter CFA piles founded in Hindmarsh Clay. Pile lengths vary, with piles at typical column piers extending to approximately 26 m to 27 m in length.

Each column is centrally located over its supporting pile, with construction tolerances in pile position accommodated at the headstock-to-column interface.

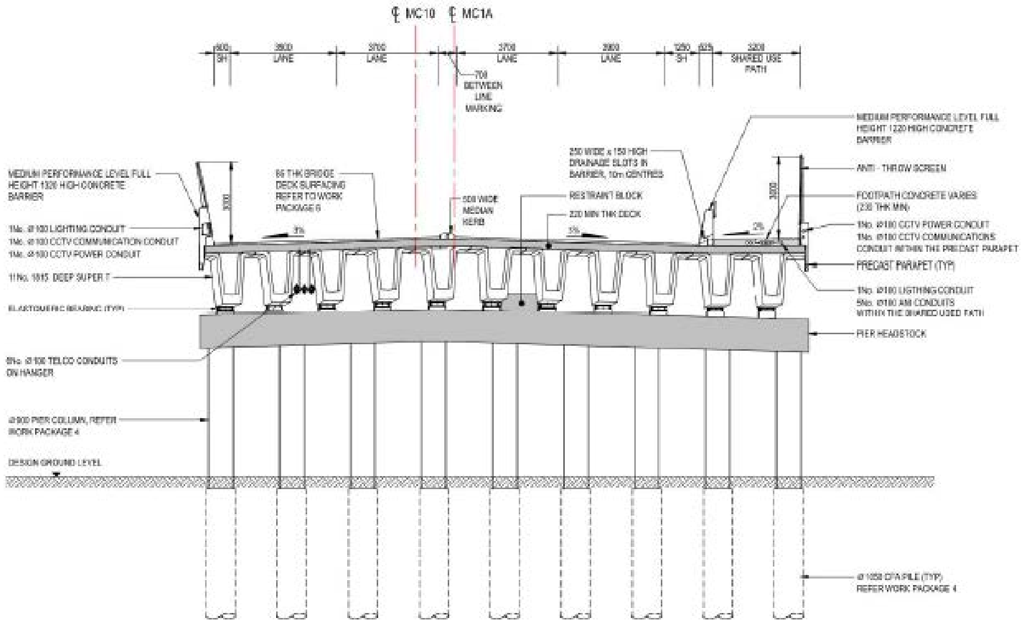

Figure 7-2: Pier 2, 3, 6, 7, and 8 typical cross section

At Piers 4 and 5, located adjacent to the rail span, 800 mm thick blade walls are used to support the headstocks. These blade piers are founded on pile caps supported by 10 CFA piles, each 1,050 mm in diameter and approximately 26 m long, founded in Hindmarsh Clay.

The blade walls are designed for robustness, with sufficient capacity to withstand rail collision loads as per AS5100 without compromising the structural integrity of the supports, as no separate deflection barriers are provided on the pier approaches (refer Section 10).

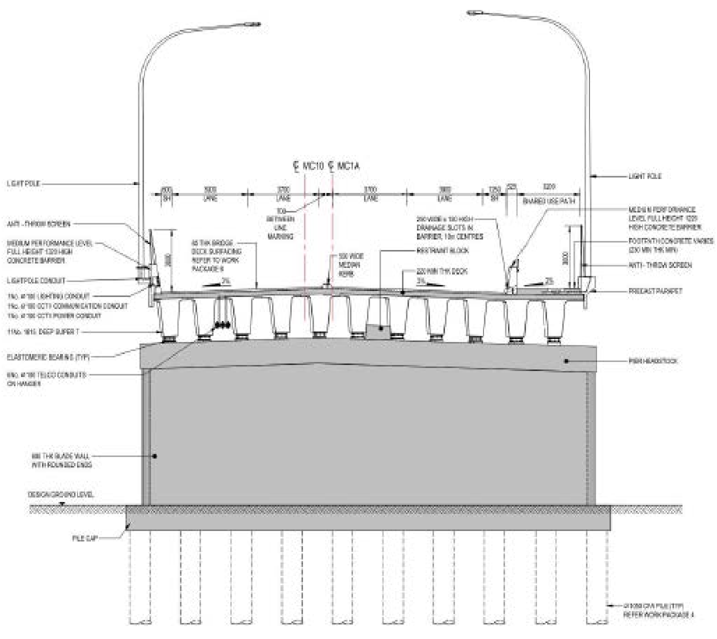

Figure 7-3: Piers 4 and 5 typical cross section

7.3.3 Bridge Bearings & Articulation

The Super T girders are simply supported on laminated elastomeric bearings, with longitudinal movements accommodated through shear deformation of the bearings. Top and bottom attachment plates are fitted with keeper plates to restrain the bearings in accordance with DIT Master Specification ST-SD-D1. Tapered steel bearing plates are provided at the top to meet DIT requirements.

Reinforced concrete lateral restraint blocks are cast between the Super T girders at each pier and abutment to resist transverse movement of the superstructure. Elastomeric bearing pads are installed along the sides of the restraint blocks to transfer transverse forces from the girders while avoiding localised stress concentrations. These restraint blocks are constructed after erection of the Super T girders.

The bridge articulation was designed to minimise the number of expansion joints, in line with DIT Master Specification ST-SD-D1. Expansion joints are located at Abutments A and B, and at Piers 3, 5, and 7, with link slabs provided between the remaining spans (refer Figure 7-5). The expansion joints are specified as Granor Ausflex AC-AR-100D strip-seal deck joints, or approved equivalent, to accommodate longitudinal movements.

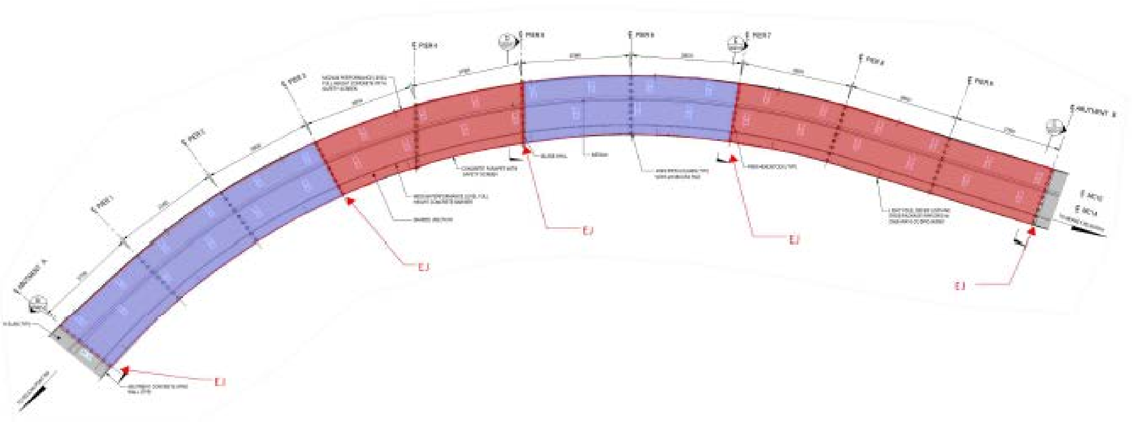

Figure 7-5: Proposed bridge articulation and deck joint locations

8.0 Super T Design in B2 Exposure Classification

The Super T girders were designed as partially prestressed members in accordance with AS 5100.5, with span lengths governed by two primary constraints:

- Precaster stressing bed capacity, limited to approximately 10,500 kN total prestressing force at lock-off
- Precasting facility lifting capacity, limited to 90 t Super T girders

Of these constraints, the stressing bed capacity was governing. Given the B2 exposure classification, the girders were also subject to stringent crack control requirements in accordance with AS 5100.5 Clause 8.6.2.3. The design therefore ensured that concrete stresses at all tendon levels remained in compression under permanent actions plus 25% of live load.

The strand layout was optimised to fully utilise the available 10,500 kN prestress capacity. In addition, actual concrete shrinkage test data was adopted in accordance with AS 5100.5 Clause 3.1.7.1 and AS 3600-2018 Supp 1:2022, removing the need to consider a ±30% shrinkage variation. This reduced tensile stresses at the lowest tendon level from approximately 0.5 MPa tension to 0.68 MPa compression under the B2 exposure classification load case, largely due to the reduction in differential shrinkage effects. These combined optimisations enabled a clear span of approximately 38 m over the 36.3 m rail corridor, allowing the structure to remain just outside the rail corridor extents.

Drying Period (days) (date)	Drying Shrinkage (μStrain)			
	Specimen 1	Specimen 2	Specimen 3	Average
7 7/08/2024	380	390	380	**380**
14 14/08/2024	480	500	500	**490**
21 21/08/2024	560	580	580	**570**
28 28/08/2024	600	630	620	**620**
56 25/09/2024	640	670	660	**660**

AS3600-2018 COMMENTARY C3.1.7.2

• Method for determining $\varepsilon^*_{csd.b}$ based on 56 day drying shrinkage test data

f'_c	65	MPa	
Test 56 day shrinkage strain	0.00066		
Test specimen t_h, $k_1(56)$	1.47		
Test specimen environment k_4	0.70		
ε^*_{cse}	0.0002	με	AS3600-2018 Eq 3.1.7.2(3)
ε_{cse} (56)	0.0002	με	AS3600-2018 Eq 3.1.7.2(2)
ε_{cse} (7)	0.0001	με	AS3600-2018 Eq 3.1.7.2(2)
ε_{cse} (56-7)	0.0001	με	
ε_{csd} (56)	0.0005	με	AS3600-2018 Eq 3.1.7.2(4)
$\varepsilon^*_{csd.b}$	0.0009	με	Rearranged AS3600-2018 Eq 3.1.7.2(5)

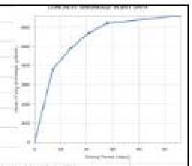

Figure 8-1: Autogenous shrinkage data obtained from 56-day testing

9.0 Seismic Liquefaction Design

As discussed in Section 4, the geotechnical assessment identified a high risk of liquefaction during a major earthquake event. This section summarises the liquefaction assessment undertaken for the Eurimbla Way Bridge and the implications of those findings for substructure design.

9.1 Lateral Spread Forces

Seismically induced liquefaction occurs when saturated, loose soils lose strength and stiffness during earthquake events and temporarily behave as a fluid. At the bridge site, the typical ground profile comprises St Kilda Formation soils, consisting of very loose sands from about 2 m depth to approximately 7.5 m, underlain by Glanville Formation soils, described as very soft to loose clay, extending for a further 4 m before transitioning to the non-liquefiable Hindmarsh Clay. The liquefaction potential was assessed for the 1 in 1,000 annual exceedance probability earthquake event, consistent with the bridge's BEDC-3 classification under AS 5100.2-2017. The assessment indicated factors of safety less than 1.0 from approximately 2 m depth, corresponding to the groundwater level, to about 10 m depth, encompassing the soft liquefiable layers. Figure 9-1 presents typical liquefaction factors of safety with depth. The geotechnical procedures used to assess liquefaction triggering are beyond the scope of this paper.

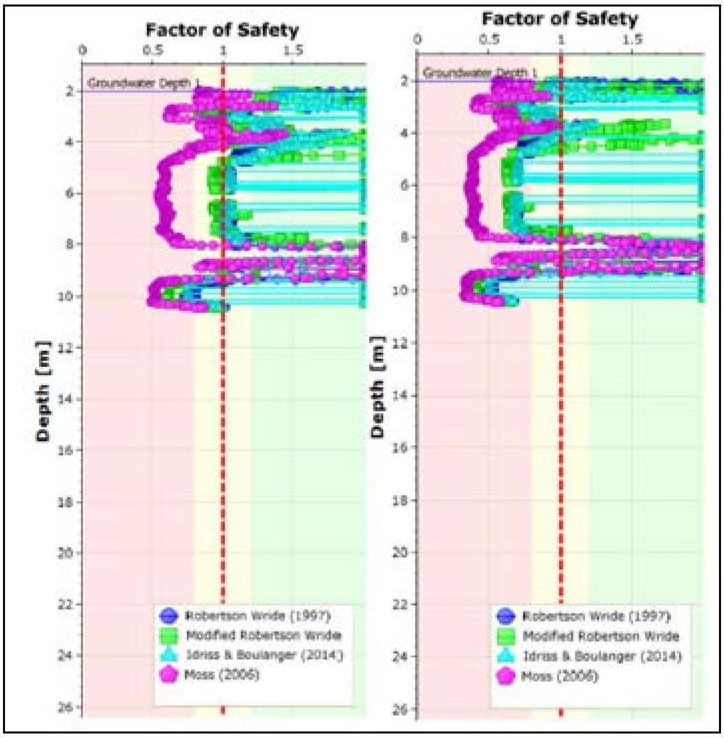

Figure 9-1: Liquefaction factor of safety for the 1 in 1,000 AEP seismic event (left: no amplification; right: 50% amplification)

Seismically induced liquefaction results in complex soil–structure interaction, in which the liquefied layers can laterally spread and impose additional demands on the bridge substructure. These actions occur in combination with inertial seismic forces and vertical dead loads, requiring the piles to mobilise resistance within the underlying non-liquefiable strata through passive earth pressures. Australian Standards, including AS 5100.3 Clause 2.8, require liquefaction to be considered but provide limited guidance on analytical methods. Accordingly, reference was made to Pile Design for Liquefaction Effects (Mitchell, 2006) and the Japanese Road Association (JRA) Specifications for Highway Bridges (2017, Part IV).

Both references broadly adopt a pseudo-static approach to account for lateral spreading effects on substructure elements. The key assumptions applied are as follows:

- Liquefied layers are taken to provide negligible lateral resistance.
- The overlying non-liquefied layer mobilises passive earth pressure against the piles.
- Liquefied layers impose a lateral pressure equal to 30% of the overburden pressure.

Section 8.3 of the JRA specification outlines the verification requirements for substructures subject to liquefaction-induced lateral spreading. The governing equations for lateral pressures in both liquefied and non-liquefied strata were adopted accordingly. A schematic representation of the analysis model is provided in Figure 9-2.

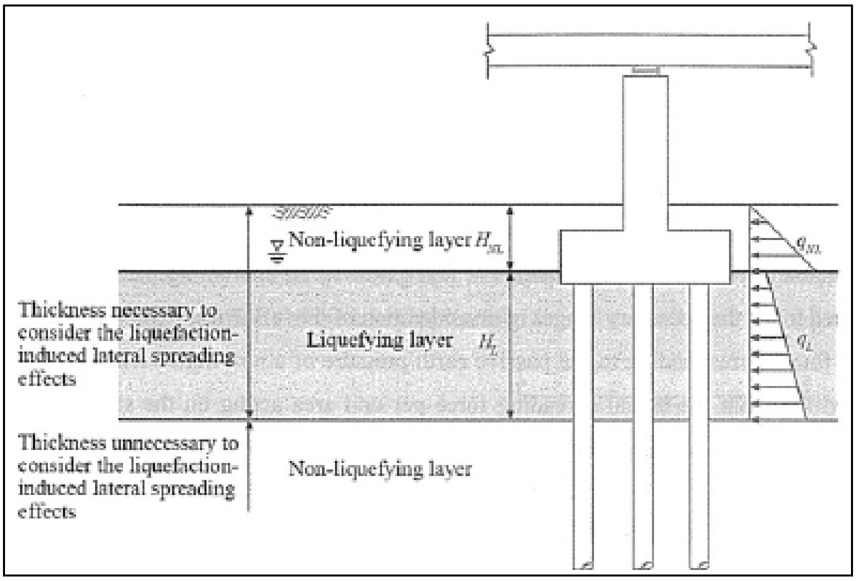

Figure 9-2: Model for calculating lateral spreading force (JRA, 2017)

$$q_{NL} = c_s c_{NL} K_p \gamma_{NL} x, 0 \leq x \leq H_{NL} \quad \ldots\ldots\ldots\ldots\ldots\ldots (JRA\ 8.3.1)$$

$$q_L = c_s c_L(\gamma_{NL} H_{NL} + \gamma_L(x - H_{NL})),\ H_{NL} < x \leq H_{NL} + H_L \quad \ldots\ldots\ldots\ldots\ldots\ldots (JRA\ 8.3.2)$$

- c_S is a modification factor relating to the distance from a water front
- c_{NL} is a modification factor for the lateral spreading forces in the non-liquefiable layer
- c_L is a modification factor for the lateral spreading forces within the liquefiable strata (0.3)
- K_p is the passive earth pressure coefficient
- γ_{NL} & γ_L represent the density of soil for both the non-liquefiable and liquefiable layers
- χ is the depth of soil under consideration

The above equations are generally consistent with established approaches for determining lateral earth pressures, with the inclusion of three modification factors. A value of C_S = 1.0 was adopted for the Eurimbla Way bridge site, reflecting groundwater levels close to the existing surface; this assumption is considered conservative relative to the JRA guidance.

The JRA standard recommends a value of C_L = 0.3, based on back-analysis of pier foundation performance during the 1995 Hyōgoken-Nanbu Earthquake, where lateral forces within liquefiable layers were observed to be approximately 30% of the total overburden pressure.

The factor C_{NL} is used to define the magnitude of passive earth pressure mobilised within the nonliquefiable layer. Where lateral spreading demands are relatively small, full passive resistance is not developed. This factor is expressed as a function of the Liquefaction Index (P_L) , determined from the integral relationship defined in the JRA provisions and outlined below.

$$P_L = \int_0^{20} (1 - F_L)(10 - 0.5x), dx \qquad \dots\dots\dots\dots\dots\dots (JRA\ 8.3.3)$$

Where;

- F_L is the liquefaction resistance factor it relates to the dynamic shear strength ratio to the seismic shear stress ratio conservatively taken as 1 at all depths as per geotechnical advice

Based on this advice the integral can be reduced to a closed form solution,P_L = 10_x - 0.25 χ^2 applicable over the depth range considered. Table 8.3.2 of the JRA standard provides corresponding C_{NL} values for given P_L results.

The resulting pressure profile was adopted as an additional load acting during the earthquake effects, in combination with the relevant soil-induced load factors of AS 5100.2-2017. The unfactored pressure distribution is presented in Figure 9-3.

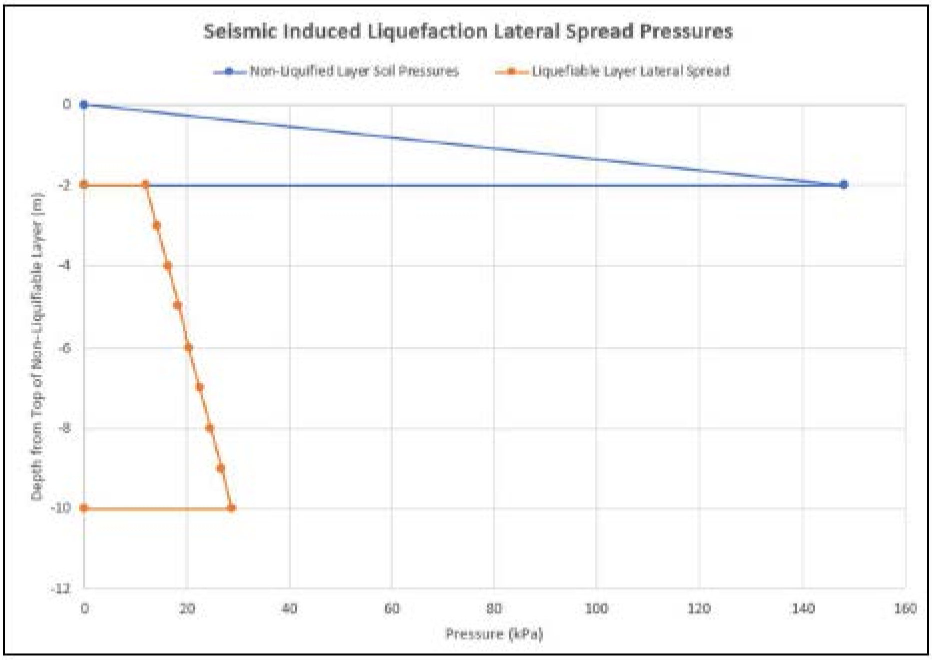

Figure 9-3: Seismically induced liquefaction lateral spreading pressures

9.2 Slenderness & Non-Linear Behaviour During Liquefaction

An important structural consideration arising from the ground behaviour described in Section 9.1 is the buckling response of piles and columns under seismic loading. The reduction in stiffness of liquefied soil layers increases the effective lengths of these elements, resulting in greater lateral deflections. This, in turn, also leads to increased non-linear P-Δ effects, with secondary moments developing about the revised point of fixity that forms during the earthquake event.

Figure 9-4 illustrates this behaviour, where G represents the supported dead load, Q represents the factored live load (which is not included in earthquake load combinations in accordance with AS 5100), and Feq represents the seismic action defined by AS 5100.

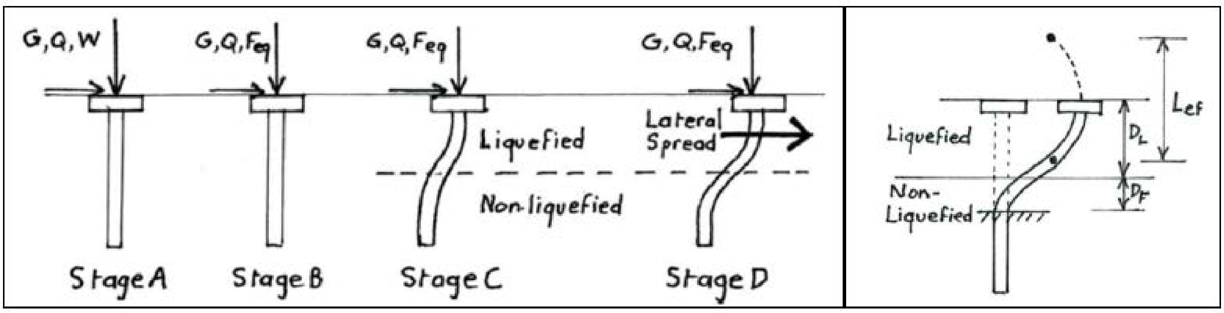

Figure 9-1: Loading for Piles During Liquefaction (Left), Buckling Lef Diagram (Right) (Mitchell, 2006)

For the Eurimbla Way Bridge, the increase in effective length of the piles and columns at several piers was significant, resulting in many elements being classified as slender members in accordance with AS 5100.5 Clause 10.3. The columns were therefore designed to account for the additional non-linear P–Δ effects, together with the moment magnification requirements of Clause 10.4.3, which are based on linear elastic analysis. The design procedure was undertaken using the relevant equations of AS 5100.5-2017.

$$\delta = \max(\delta_s, \delta_b) \quad \ldots\ldots\ldots\ldots\ldots\ldots (AS5100.5\ Cl\ 10.4.3)$$

$$\delta_s = 1/(1 - \frac{\sum N^*}{\sum N_c}) \quad \ldots\ldots\ldots\ldots\ldots\ldots (AS5100.5\ Eq\ 10.4.3(1))$$

Where;

- N_c, the buckling load is determined through the below equation

$$N_c = \left(\frac{\pi^2}{L_e^2}\right)\left(\frac{182 d_o(\varphi M_c)}{(1+\beta_d)}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (AS5100.5\ Eq\ 10.4.4)$$

The impact of liquefaction and the resulting behaviour above was found to be significant. For the critical pier, the resulting increase in maximum bending moments on the pile and column were found to be 25% and 20% respectively.

9.3 Summary

Liquefaction was a critical design consideration for the Eurimbla Way Bridge because it materially increased the demands on the bridge substructure during the seismic design event. The loss of soil stiffness within the liquefiable layers reduced lateral support to the piles and columns, increased effective lengths, and amplified bending moments through lateral spreading and second-order P–Δ effects. As a result, liquefaction was a governing factor in the design of several foundations, influencing both the pile lengths required to achieve adequate fixity in competent non-liquefiable strata and the reinforcement detailing needed to resist the increased seismic actions. This assessment demonstrates that, for bridges founded in soft coastal soils, liquefaction can govern substructure design.

10.0 Removal of Deflection Walls

10.1 Requirements of a Deflection Wall

The reference design for the Eurimbla Way Bridge incorporated deflection walls at the approaches to the piers adjacent to the four-track ARTC rail corridor. Given the very soft ground conditions, preliminary assessments indicated that these walls would require extensive deep foundations to resist collision loads in accordance with AS 5100.2. This presented a significant challenge in both cost and constructability.

AS 5100.1-2017 does not explicitly mandate deflection walls, instead it recommends that their provision be determined through a project-specific risk assessment. Clause 15.3.4 (non-frangible piers) confirms that the final requirement is subject to approval by the relevant rail authority.

15.3.4 *Piers other than frangible piers*

Piers shall be designed for the collision loads specified in Clause 11.4.2.3 or Clause 11.4.2.4 of AS 5100.2, and shall comply with the geometric requirements of this Clause.

Piers shall be protected from head-on impact from derailed trains by deflection walls unless approved otherwise by the relevant authorities.

Figure 10-1: Excerpt from AS5100.1-2017 Clause 15.3.4

Similarly, ARTC Specification ETS-09-00 permits alternative solutions where justified by a suitable risk-based approach.

9.3.4 Collision from Rail Traffic

9.3.4.1 New bridges

Unless otherwise approved by ARTC, the design of all the new bridges shall comply with all the minimum requirements of AS 5100 for the protection against rail traffic collision loads in conjunction with the following additional requirements:

1. As per AS 5100, all new bridges over tracks shall be designed to:
   a. Avoid collapse of structures over rail due to impact from derailed trains.
   b. Reduce the severity of impacts with structures to reduce the probability of injury to occupants of derailed trains.
2. A site-specific risk assessment acceptable to ARTC shall be carried out to determine the requirements for protection of bridge piers, abutments and the ends of the through girder, arch or truss.

Figure 10-2: Excerpt from ARTC ETS-09-00

Accordingly, a detailed assessment was undertaken to determine whether the deflection walls could be safely omitted while maintaining compliance with the applicable standards and stakeholder requirements.

10.2 Maximising Horizontal Clearance

The primary mitigation measure adopted was to maximise the span length over the rail corridor (Span 5), thereby increasing the horizontal offset between the track centrelines and the bridge substructure.

In conjunction with span optimisation, the dimensions of the blade walls and headstocks were minimised within the constraints of AS 5100.1 and the rail clearance requirements. This resulted in clear distances of approximately 6.5 m to the western ARTC track and 5.6 m to the eastern track (refer Figure 10-3).

These increased offsets formed the basis of the subsequent risk assessments and were critical in reducing the likelihood of train-to-pier impact.

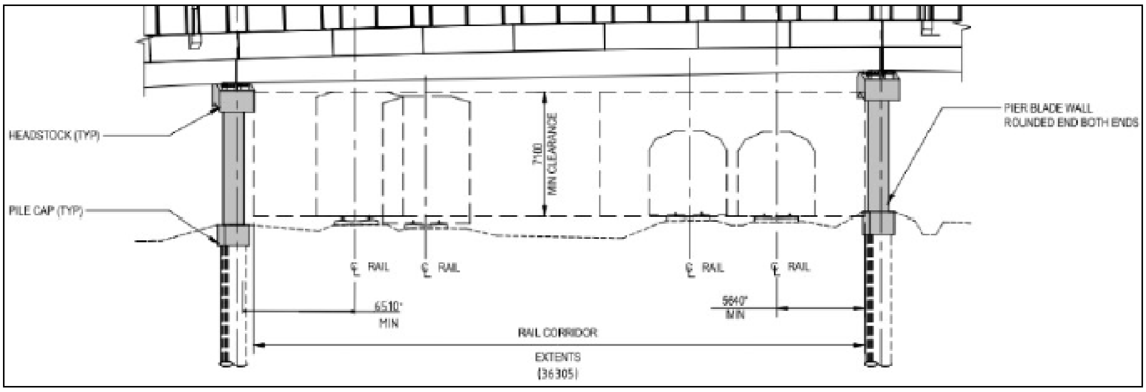

Figure 10-3: Rail Corridor Clearances to Supports

10.3 Deflection Wall Risk Assessment

To verify the adequacy of the optimised arrangement, two independent risk assessment approaches were undertaken:

- a probabilistic assessment in accordance with International Union of Railways (UIC) 777-22, and
- a rational, first-principles-based assessment using ARTC parameters.

The probabilistic method in UIC 777-2, an international guideline for structures built over railway lines, is based on two probability components: the likelihood of a train derailing on approach to the bridge (P1) and the likelihood of a derailed train striking a bridge pier (P2). The combined probability of impact is taken as the product of these two terms. Both probabilities are defined using the relationships provided in UIC 777-2 Appendix F:

$$P1 = e_r * d * Z_d * 365 * 10^{-3}$$

$$P2 = \left\{ \left[\frac{b-a}{b}\right]^2 + \left[\frac{b-(a+4.2)}{b}\right]^2 \right\} * 0.25 * \frac{c}{d}$$

Where;

- e_r is the derailment rate for trains per train kilometre (Rail Safety Report 2022-2023 by ONRSR)
- d is the longest derailment path in metres
- Z_d is the number of trains per day
- a, b and c are geometric values relating to clearances, derailment speeds and derailment paths

Figure 10-4 presents the relationship between the probability of impact and the offset distance from the track centreline to the substructure, derived using this method. Results are shown for freight trains travelling at 30 km/h (posted speed) and 60 km/h (posted speed with a factor of 2 applied to account for potential future increases), assuming an unchanged daily train frequency.

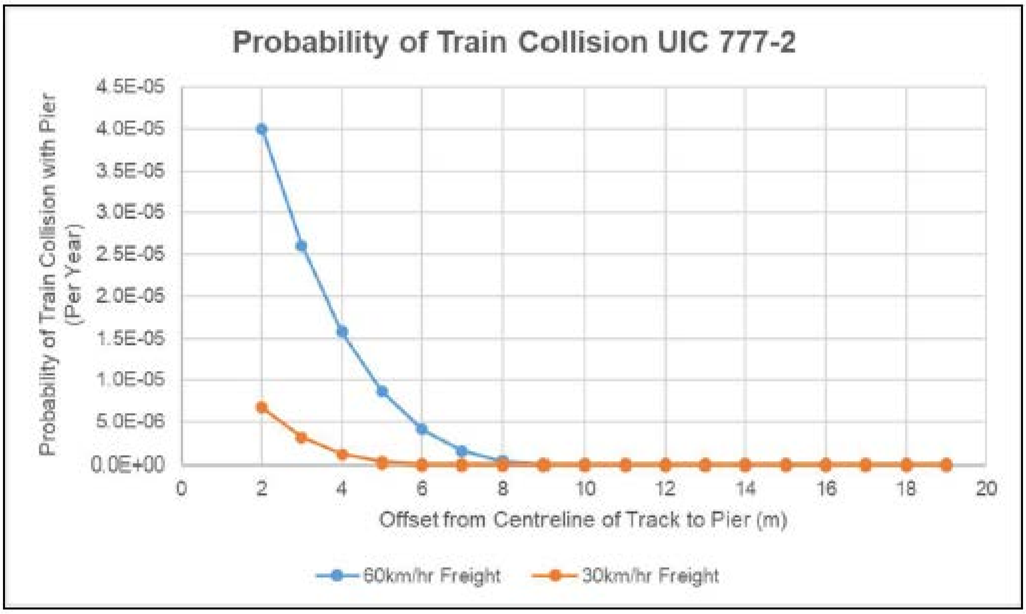

Figure 10-4: Comparison of probability of train collision versus pier offset using UIC 777-2

The second method for assessing the risk of train collision with the bridge piers adopted a rational, first-principles approach using parameters defined in ARTC Standard ETS-09-00. While the specification primarily provides parameters intended for determining head-on collision forces, the same parameters were considered suitable for estimating the likelihood of a derailed train reaching the substructure.

A train speed of 60 km/h was adopted to account for potential future increases above the posted speed. The following parameters were taken from ETS-09-00 Clause 9.3.4.1 for the assessment:

- Freight train mass (m) = 1000T
- Derailment angle = 3.5°
- Deceleration = 3ms-2
- Train width from rail centreline = 2.5m (ETS-07-00)

Using the parameters listed above and the third kinematic equation of motion, the maximum lateral deviations for each track were calculated:

$$v_0^2 = u^2 + 2as$$

Where;

- v_0 is the final velocity in ms^{-1}
- u is the initial velocity in ms^{-1}
- a is the acceleration ms^{-2}
- S is the total displacement in m

The results of the two risk assessments are shown in Table 10-1 below.

Table 10-1: Rational Method and UIC 777-2 Results

Track	Operational Speed (km/h)	Clearance from Closest Pier (m)	Rational Method Maximum Lateral Deviation (m)	UIC 777-2 Probability of Collision (ARI)
ARTC Western Track	60	6.51	5.34	1 in 392 964
Private Rail Eastern Track	60	5.64	5.34	1 in 182 877

As shown in Table 2, the rational method indicates a maximum lateral train deviation of 5.34 m, confirming that a derailed train would not reach the pier under the adopted parameters.

By comparison, the UIC 777-2 method results in a calculated average recurrence interval (ARI) of approximately 182,877 years, which is significantly greater than the ultimate limit state (ULS) design event typically adopted for bridges in accordance with AS 5100 (approximately 1 in 2,000 years). This disparity arises primarily from the differing approaches used to estimate lateral train displacement. The UIC method adopts an empirical relationship of the form $V^{0.55}$, where V is the train speed at derailment. This relationship is based on statistical regression of historical data and does not explicitly account for the site-specific conditions at the Eurimbla Way Bridge.

In contrast, the rational method is based on first principles and explicitly incorporates site-specific parameters to define derailment trajectories. On this basis, despite the very high ARI indicated by the probabilistic method, the rational approach was adopted as the governing basis for the deflection wall assessment.

Following this assessment and agreement with the rail authority, it was concluded that a head-on collision with the piers adjacent to the rail corridor was not a credible scenario. Deflection walls were therefore not required, provided that the following criteria were satisfied:

- The blade wall piers are designed for robustness in accordance with AS 5100.2 Clause 11.4.2.3 collision loading requirements
- The blade wall headstock is designed to support the superstructure with a 1m cantilever from the edge of the blade wall

Relative to the reference design incorporating deflection walls, this approach resulted in significant savings in construction and material costs, primarily associated with the elimination of the large deflection walls and their associated deep foundation works.

11.0 Conclusion

The Eurimbla Way Bridge demonstrates the value of an integrated structural and geotechnical design approach, supported by contractor input, in delivering infrastructure within a constrained rail corridor environment.

A key project driver was the requirement for an accelerated construction programme. This was achieved through a predominantly precast concrete solution combined with an optimised span and structure arrangement, which reduced reliance on extensive ground improvement works. As a result, the 380 m long, 10-span bridge was delivered in approximately 12 months, exceeding the initial programme expectations.

Geotechnical and seismic considerations, including liquefaction and P–Δ effects, were addressed using a combination of international guidance and first-principles assessment methods. This approach enabled a robust substructure design despite the limited availability of detailed local guidance. In parallel, a risk-based assessment of derailment impacts demonstrated that deflection walls were not required. Their removal, together with span optimisation, delivered significant savings in construction cost and programme while maintaining safety and compliance with the relevant standards.

Overall, the project highlights the effectiveness of value-driven design decisions in achieving safe, efficient, and constructable bridge solutions within constrained rail corridor environments.

12.0 References

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14.0 Author Biographies

Angelo Maio is a Structural Engineer at SMEC Australia Pty Ltd working in their Bridges & Structures team located in Adelaide, South Australia. Angelo has over 3 years of experience in the detailed design, verification, assessment and construction support of road, rail, and pedestrian bridges. He is the structural engineer responsible for producing detailed structural analysis and design calculations for the substructure and superstructure elements of the Eurimbla Way Bridge.

Kin Yap is a chartered structural engineer in SMEC Australia Pty Ltd specialised in rail and road bridges with over 18 years industry experience. He has extensive experience in prestressed concrete & steel bridge design, bridge widening & strengthening design, retaining walls & foundations design, as well as experienced in numerous site inspections and level 2 condition assessment for existing bridges and structures. Kin is the structural design lead of the Eurimbla Way Bridge.

Author

Angelo Maio, Bridge Engineer
Kin Yap, Team Leader, Bridges & Structures