

Port River Expressway – Stage 1 - Bridge Over Eastern Parade.

Peter Selby Smith CPEng FIEAust
Project Engineer- Bridges
GHD Pty Ltd
380 Lonsdale St
Melbourne, 3000
Australia

Graeme Joynson
Senior Engineer- Bridges
GHD Pty Ltd
380 Lonsdale St
Melbourne, 3000
Australia

Zachary McGain MIEAust
Structural Engineer- Bridges
GHD Pty Ltd
380 Lonsdale St
Melbourne, 3000
Australia

ABSTRACT

The Port River Expressway Project will provide a four-lane road linking the Port of Adelaide with the Salisbury Highway.

Stage 1 involves the road works component extending 4.7 km from east of the port to South road. The project includes provision of a bridge over Eastern Parade and the railway to the west of the intersection.

The bridge is a nine span continuous structure, crossing the railway and eastern parade at a 40 degree skew. The western abutment and the first two piers at the western end are unskewed. It has seven internal spans of 32 metres and end spans of 27.2 metres making a total length of 278.4 metres.

The deck has an overall width of 18.2 metres, which accommodates 2 eastbound lanes and 2 westbound lanes, separated by a 1.0 metre wide central median.

The superstructure consists of four steel box girders with an in-situ concrete slab. The piers and abutments are of reinforced concrete and are founded on bored piles.

The presence of aggressive soils and the possibility of acid sulphate attack has led to additional protective requirements in the components in contact with the existing soil strata identified as the St. Kilda formation. This includes the piles, pile caps and bases of retaining walls.

Ground improvement techniques were required on the approaches, including use of stone columns and preloading.

Keywords: steel box girders, in-situ concrete slab, acid sulphate, durability, stone columns.

1. Introduction

The Port River Expressway will link the Port of Adelaide with major road and rail transport corridors. It will create a direct route for heavy traffic, reducing travel times, and taking these vehicles off suburban streets.

The project is to be completed in three stages. Stage 1 involves construction of 4.7 km of 4-lane road between Eastern Parade and South Road, including a major bridge over Eastern Parade.

Stages 2 and 3 consist of bascule bridges over the Port River for both road (Stage 2) and rail (Stage 3) traffic.

The contract for Stage 1 was awarded to Bardavcol in October 2002 with York Civil appointed as subcontractor for the bridgeworks.

GHD was engaged to design the bridge over Eastern Parade.



Figure 1: location Map

1.1 Eastern Parade Bridge

The new bridge carries the Expressway over Eastern Parade and an adjacent parallel railway spur. The bridge is 278 m long, 18.2 metres wide, carrying two lanes of traffic in each direction. It consists of nine spans - seven at 32 metres length, and two 27 metre end spans.

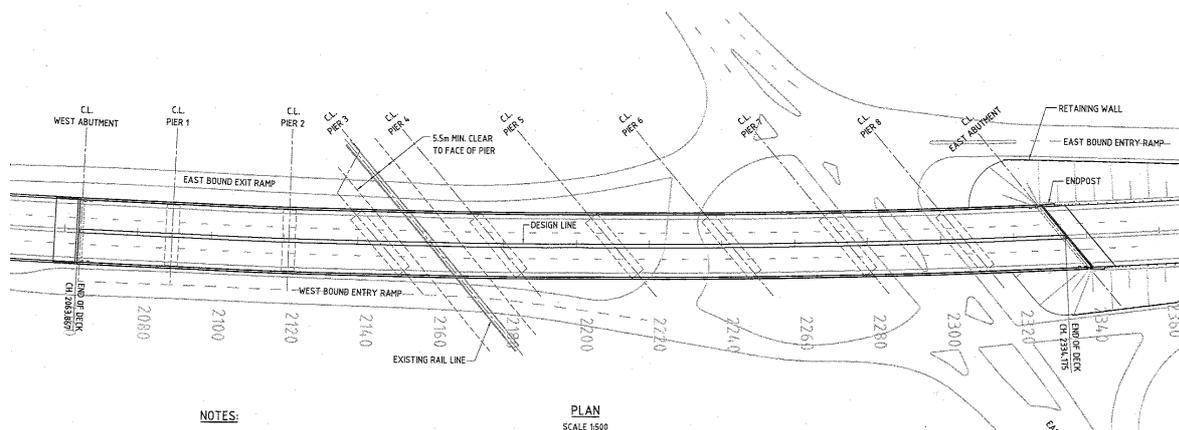


Figure 2: Bridge layout

Poor founding conditions at the site led to the adoption of ground improvement techniques. Embankment height was to be kept to a minimum (and hence the bridge deck needed to be shallow in depth).

The four-lane bridge consists of a four single box girders with composite concrete deck. A novel technique for joining the steel boxes at the piers eliminated the need for site welding of the splices and simplified erection of the spans.

Durability was an important consideration due to the close proximity of the ocean, and acid sulphate soils. Measures were taken to protect each element of the structure both above and below ground.

2. Design Parameters

The structure was designed to the existing Austroads Bridge Design Code ⁽²⁾, except that the new draft standard (DR 00375) ⁽³⁾ was used for the loading. This included the new SM1600 loading, seismic considerations, and new traffic barrier loadings.

2.1 Structural Concept

The poor founding conditions governed the selection of span length and embankment height. They also influenced the selection of a steel superstructure, which reduced the dead load of the completed structure.

The balance between the length of the bridge and the ground improvement was chosen to optimise the overall cost.

The relative high cost of the ground improvement at the western approach, using stone columns, meant that it was economical to increase the length of the bridge by two spans, reducing the embankment height to about 4 metres.

At the eastern end the conditions were better with only preloading required to support a 7 metre embankment height.

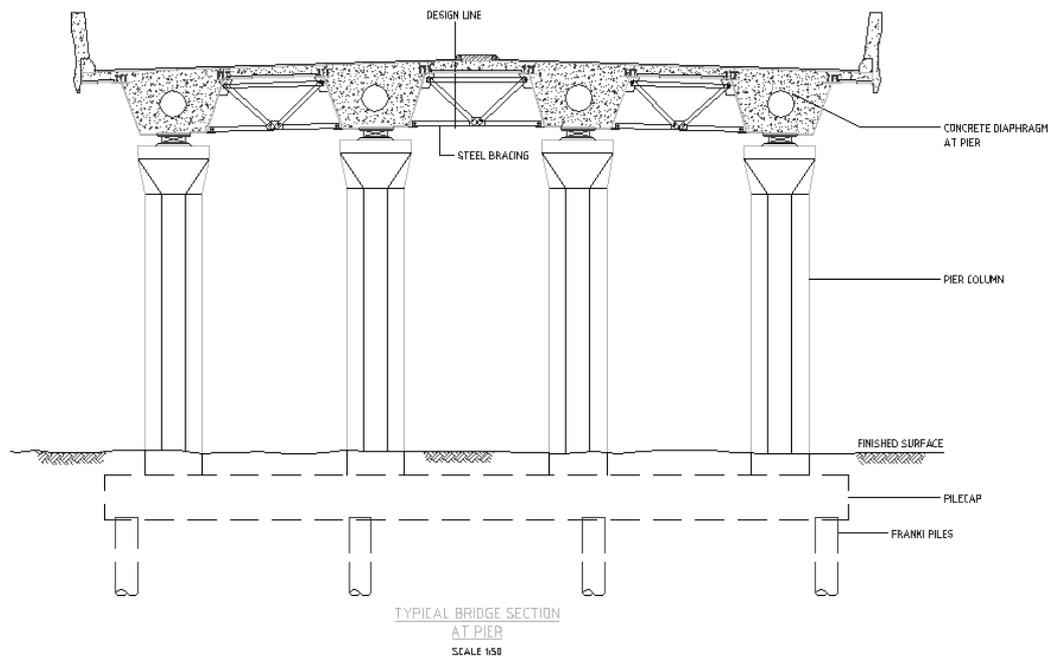


Figure 3: Typical Bridge Cross section

The bridge superstructure consists of 4 steel box girders, 2.25 m wide by 1200 deep with a composite in situ deck (refer fig. 3). The bridge is continuous, with 32 m internal spans and 27.2 m end spans for a total length of 278.4 m. The bridge has a 40 degree skew at the Eastern Abutment to pass over Eastern Parade and the parallel railway line The Western Abutment and the adjacent 2 piers are constructed normal to the design line which follows a 2600 metre horizontal curve.

The girders are supported at each pier, on four independent octagonal columns (refer fig. 5). A pile cap distributes load to the piles below.

The box girders are spliced at the piers using an in situ concrete diaphragm. Bridge barriers are medium containment level, but are 1300 mm. high to cater for cyclists.

Due to its location, the site had significant durability issues and a formal review was undertaken by in-house materials specialists to identify the major issues, and measures to be incorporated in the design to deal with them.

2.2 Durability Considerations

The client required a functional life of 100 years.

The Eastern Parade Bridge is located approximately 3 km from the coast. Saline wetland is located to the North East and the tidal Port River is located approximately 0.5 km west of the bridge site.

The proximity to the coast may give a slightly elevated concentration of atmospheric chlorides that were considered when assessing the performance of protective coatings on the steelwork.

With respect to the below ground microenvironment, the soil along the proposed expressway alignment is situated in an area of an Estuarine Muds and Sands profile. At the top of this profile (at 2-5 metres depth) is the St Kilda formation, which is known to be highly aggressive in terms of salinity and acid sulphate potential.

The concrete piles, which the bridge is founded on, penetrate this highly aggressive material. Pile caps also found on this material and are also likely to be exposed to fluctuations in the water table.

Austrroads ⁽²⁾ does not provide specific guidance for piles in aggressive soils, with an indicated exposure classification U and so guidance from AS 2159 ⁽⁴⁾ for generally a 40-60 year design period was adopted. Given the 100 year design life, it was considered that the classification rating be increased to very severe.

Measures adopted included:

- Concrete strength of 50 MPa,
- Supplementary cementitious materials (25-30 % Fly-Ash) were used to provide a concrete mix with high sulphate resistance,
- 70 mm. cover to reinforcement,
- A low water cement ratio,
- Waterproofing the concrete.

To achieve the last requirement, a hydrophobic pore blocking waterproofing admixture was used in the pile concrete. The close proximity to the surface of the pile caps made cheaper options available, and a sprayed waterproof bituminous membrane was applied to this sides.

For structural concrete components above the ground the atmospheric exposure is relatively benign and satisfaction of the requirements of B1 Exposure Classification of Austrroads Bridge design codes ⁽²⁾ were deemed to provide adequate protection. Generally an addition 5 mm. cover was adopted above these requirements.

For the structural steel the durability of the components depend on the applied protective coating system.

Painting specifications were designed to ensure compliance with the minimum requirements of AS 2312. The environment was classified as “Category C: Medium”

Coating options considered were:

- Inorganic Zinc Silicate applied in a single coat,
- Zinc Rich Primer / High Build Epoxy / Poly urethane.

Both these options have an expected service life of 25-40 years.

Only a limited colour range was available for the first option and painting over it reduces its service life. The client also had a requirement, based on local experience, for additional paint thickness to be applied on the girder soffit. This led to the second option being the only compatible solution and was finally adopted.

2.3 Superstructure Design

Modelling of the superstructure was undertaken using a shear flexible grillage due to the propensity of the box girder to distort under differential loading. Careful assessment of the model parameters was undertaken using beam and plate finite element models to calculate shear area and torsion constants for the various elements in the model.

2.3.1 Torsion Constant

The torsion constant was determined by comparing several methods from Hambly⁽¹⁾ to verify the value obtained. These included the shear flexible grillage, a ‘spine-beam’ supported grillage model, and a FE model. Reasonable agreement between these models gave confidence in the chosen parameters.

The FE plate model of a section of the box was created using transformed section properties for the composite deck. The purpose was to examine the effects of warping and distortion, which could not be modelled otherwise. The spacing of the internal box bracing was determined to minimise these effects in this model and the parameters applied to the others. It is worth noting that the value obtained using this model was found to give an answer slightly higher than from the St. Venant equation for a thin walled section, but of the same magnitude..

It was also noted that a model used to determine the torsion constant must have one end fixed so that it remains plane after application of the torsion, otherwise the value obtained can be greatly reduced by warping in the box.

The torsion constant was halved and applied to the boxes in both directions in accordance with the methods described by Hambly⁽¹⁾.

2.3.2 Shear Area

The distortion of the box under differential loading is taken into account by the shear area of the transverse grillage members between the webs of the box girder. Due to the spacing of the bracing, it was necessary to calculate a number of shear areas and apply these to represent the different distortion characteristics along the length of the box.

Both the Torsion Constant and Shear Area parameters were important in achieving an efficient design. They are parameters that can greatly affect the distribution of the load between the boxes. It is also important that these parameters accurately reflect what has been included in the design and vice versa. In this instance, the shear area of a box section with a cross brace was in the order of 1000 times larger than without. Consequently, the cross bracing at these locations had to be substantial in order to carry the calculated load which was attracted to it in the model.

2.3.3 Girder Sections

Two distinct box girder sections were adopted – one for the mid span area and the other for the negative bending region over the piers. In the latter section, the deck concrete was assumed to be cracked, and its contribution to model stiffness and strength ignored.



Figure 4: Typical Box Girder under construction

2.3.4 Transfloor Formwork

Precast “Transfloor” panels were used as lost formwork for the deck slab. The panels were 60 mm. thick and supported an in situ 160 mm. slab. Because the panels were to be discontinuous at short intervals along the length of the deck, they were ignored in the concrete compression block used to determine the ultimate positive moment capacity. It was subsequently found that this had very little effect on the result.

The transfloor panels were originally proposed to run continuously across the deck in 2.5 metre strips with a gap at each flange support. The top flange shear studs were to be fused to the on site, after the placement of the transfloor, to avoid conflict with the trusses and other panel reinforcement.

It was later found that the fusing of the shears studs would affect the protective coating on the underside of the top flanges, and so the studs had to be installed in the workshop before the painting. This resulted in the transfloor panels having to be made simply supported between the girder flanges to avoid traversing the studs.

The panel strength required a corresponding upgrade for increased positive bending moment, and re-orientation to align with the bridge centreline in 7.5 metre lengths.

2.3.5 Pier Connection

The steel box girders are joined at the piers using a 1200 mm long reinforced concrete “plug” diaphragm. This detail avoided the use of site welded splices in the girders and made construction easier.

To achieve this, large bending and shear stresses needed to be adequately transferred between the concrete diaphragm and the webs and flanges. Additional shear studs provided the

continuity, and careful attention was given to the stress paths involved to ensure adequate strength was achieved at both Serviceability and Ultimate Limit States.

Fatigue of the components due to live load cycling was also considered and found not to be a concern.

Multiple layers of reinforcement at the top of the section provided the necessary tensile capacity over the gap in the girders.

A 300 mm thick concrete slab in the bottom of the girders extending 2.5 metres either side of the diaphragm ensured gradual dispersion of the compression forces in the diaphragm to and from the bottom flange and webs of the girders. A transverse RC beam within the end of this slab was designed to resist the eccentricity of the horizontal forces, preventing a local kink.

2.4 Substructure Design

The substructure was modelled using a simple grillage model of the superstructure, with only basic torsional characteristics input into the model to determine the loading of the substructure. This was proven to be adequate for the design of the substructure with only small deviations from the reactions from the final superstructure modelling.

2.4.1 Articulation

Due to the length of the bridge, thermal movements are large. Expansion joints were to be limited to the abutments and this meant that finger joints were required. It was decided to fix the bridge closer to one abutment so that only one finger joint would be required.

As the eastern abutment is skewed, it was decided to locate the finger joint at the other end, which is square, to avoid unnecessary complication. To achieve this Pier #6 (of #8) was fixed, allowing a simple rubber seal type joint at the skewed end.

2.4.2 Piles and Pile Cap

As the ground conditions were extremely poor, the piles were originally given an Ultimate Limit State capacity of 1100 kN. Due to this low value it was decided to model the pile caps using a finite element model to accurately account for the flexibility of the pile cap. In coming to this decision, models using grillage, spine beam and the finally adopted brick model were prepared.

It was found that the grillage model tended to underestimate the pile loads, and this was put down to the indirect load path to the pile. The spine beam model was found to overestimate the pile load, which given the already low pile capacity, was considered to be unacceptable. It was decided to proceed with the FE brick model. Extreme fibre stresses were used to back calculate the moment.

This approach was found to be quick and using an approximate strut and tie check, also to result in sensible reinforcement quantities. While not a high-end application of FEA, in this application it was found to be a useful tool, which allowed quick assessment of options.



Figure 5: Completed piers

Originally it was proposed to use precast 350 mm square driven piles of about 20 metre length.

The piling subcontractor offered a saving using 500 mm diameter bored enlarged base “FRANKI” piles to 15 metre depth, and these were eventually adopted.

3. Advantages and Disadvantages of the Box Girder Design

The advantages of this type of structure are that it is:

- Light weight - therefore erection is easier and loads on the substructure are reduced which is a significant advantage on sites with poor foundation conditions,
- Easier to erect with no site welding,
- Better load distribution compared to I girders due to torsional rigidity leading to a more economical section,
- Does not require external cross bracing other than at supports,
- Aesthetically more pleasing than I girders,
- Competitive with longer Super-T spans.

The disadvantages are:

- Expense of fabrication relative to I beams – generally fabrication will be more expensive than a simple I-beam, but was found to be less than the cost of the extra material,
- A significant amount of additional shear studs are required to affect the pier joint, but these are relatively cheap and easy to install.

4. Conclusion

Modelling of the structure posed some unique challenges that were overcome to produce an efficient and attractive structure.

The box girder arrangement has several advantages over Steel I girders and precast concrete Super-T beams that should make them an attractive option for future projects.

The box girder section is a technically better solution. It gives a more efficient distribution of loads transversely across the bridge and therefore lighter sections.

The box girders have been found to be more economic than I-girders. Their lesser section area outweighs the additional fabrication costs.

The lack of external bracing on the box girders, apart from at supports, makes easier to maintain and repaint the exposed external surfaces.

Utilising the skills of specialist materials technologists for this job enabled alternatives to increased cover and higher strength concrete to be used with good effect to improve durability. This is an increasing area of concern for bridge owners, with more emphasis being placed on durability considerations.

5. Acknowledgements

Owner	Transport South Australia
Main Contractor	Bardavcol
Bridge Subcontractor	York Civil
Structural and Durability Design	GHD Pty Ltd

6. References

1. HAMBLY E.C., "Bridge Deck Behaviour", CHAPMAN & HALL, Second Edition 1991
2. AUSTROADS, "92 Austroads Bridge Design Code", KELVIN PRESS, Sydney 1992
3. STANDARDS AUSTRALIA, "Bridge Design- Part 2: Design Loads", STANDARDS AUSTRALIA DR00375 (Draft Australian Standard for Comment), Sydney, Australia, November 2000, 63 pp.
4. STANDARDS AUSTRALIA, "AS 2159 Piling - Design and Installation", STANDARDS AUSTRALIA, Sydney 1995.
5. Port River Expressway Project website:
http://www.transport.sa.gov.au/transport_network/projects/port_xpress/index.asp