Eastlink Project Steel Composite Box Girders over Princes Highway

Andrew Gallagher B.Eng CPEng RPEQ, Parsons Brinckerhoff
Andrew is a Principal Structural Engineer with PB. He has over 15 years of postgraduate experience in infrastructure design and construction. On Eastlink Andrew led the design team for the steel bridges on Princes Highway interchange. He is currently structural team leader for the northern surface works on Brisbane’s North South Bypass Tunnel. Other recent projects with PB include Eastlink, Lane Cove Tunnel, and the Auckland Central Motorway Junction.

Contacts
Phone  07 3218 2266
Fax    07 3831 4223
Email  agallagher@novex.com.au

Hank Sargent B.Eng RPEQ, Parsons Brinckerhoff
Hank is a Senior Structural Engineer with PB. He has 11 years experience on infrastructure, hydraulic, civil and structural design projects. On Eastlink Hank undertook the project management of the Princes Highway Interchange bridge design team. Hank’s other recent experience includes being on the tender design team for the main span of Brisbane’s Gateway Upgrade Project.

Dr Aaron Yuan B.Eng Phd, Parsons Brinckerhoff
Aaron is a structural engineer with PB. His post graduate research was specific to steel structures. His skills lie in advanced structural analysis and design, especially with nonlinear and instability issues. His work experience is predominantly in bridges, marine structures, mining infrastructures, and construction solutions.
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SYNOPSIS
The Princes Highway Interchange is part of Melbourne’s “Eastlink” Project – Australia’s largest infrastructure project currently being constructed by Thiess-John Holland Joint Venture. The three level interchange includes twin structures to carry each carriageway of Eastlink over the Princes Highway, and a third level structure to carry Heatherton Road over the interchange below.

Steel structures have been used to significant advantage to achieve spans up to 65m using continuous steel composite box girders. The light weight steel girders offer significant advantage in that the spans were erected by crane over the Princes Highway and spliced together using field-bolted connections. Design of these structures was in accordance with the AS5100 Bridge Code, in particular AS5100.6 Steel and Composite Construction.

This paper provides a summary of the design issues faced in developing the details for medium span steel composite box girder bridges. Some of the innovative design approaches adopted are presented, including the use of composite top and bottom flange concrete or “double composite” design. The methodology for web plate design, approach to the development of structural details, and the various analysis methods that were adopted for fatigue assessment, and stage analysis are also presented. Other design components included the development of a precast concrete pier solution and post tensioned concrete headstocks. Seismic assessment and dynamic analysis was also undertaken.

1 INTRODUCTION

The Princes Highway Interchange on Melbourne’s Eastlink Project is a three level interchange. Structures included the Princes Highway Bridges (Eastlink Motorway over Princes Highway) and the Heatherton Road Overpass (Heatherton Rd over
Eastlink Motorway and Princes Highway). All structures were of steel composite construction and are described below.

1.1 Princes Highway Bridges

The Princes Highway Bridges are twin carriageway structures carrying the northbound and southbound Eastlink carriageways over the existing Princes Highway. Features of the structures include the following:

- Three lane bridges plus shoulders;
- Twin two span bridges with maximum 50m span over Princes Highway;
- Precast concrete piers;
- Three (3) No. 2m deep steel composite box girders per span;
- Concrete composite deck with precast formwork system;
- Concrete internal diaphragms and “double composite” flanges over the piers.

The typical cross section is illustrated below.

Figure 1.1 - Princes Highway Bridge Section

1.2 Heatherton Road Overpass

The Heatherton Rd Overpass is a single carriageway structure carrying Heatherton Road over the Princes Highway bridges and the existing Princes Highway. Features of the structure included the following:

- Two lane bridge plus footpaths both sides;
- Four span structure with maximum 65m span over Eastlink;
- Post-tensioned pier headstocks;
- Four (4) No. 2.5m deep steel composite box girders per span;
• Concrete composite deck with precast formwork system;
• Concrete internal diaphragms and “double composite” flanges over the piers.

The typical cross section for Heatherton Rd Overpass is illustrated below.

![Figure 1.2 - Heatherton Rd Overpass Bridge Section](image)

## 2 DOUBLE-COMPOSITE STEEL BOX DESIGN

“Double-composite” steel girder design is an innovative solution that incorporates a composite concrete bottom flange in the negative moment regions of steel plate girder or steel box girder continuous bridges. For the Eastlink Princes Highway Interchange bridges, this method was introduced during the detailed design phase and offered the following significant benefits:

- The double composite concrete compression flange allowed a significant reduction in steel plate thickness leading to savings in supply and fabrication;
- The lifting mass of the girder segments was reduced allowing longer segments to be erected using conventional methods;
- The “double composite” action allowed a compact section to be reasonably achieved in the negative moment region due to the lowering of the plastic neutral axis, and the reduction of the depth of the web in compression.

By achieving a compact section in accordance with AS5100.6 Article 5.1.3, the design section capacity can be based on the development of the full plastic moment before the onset of local buckling of the web or bottom flange in compression. A comparison of the plastic moment design section over the pier for the Heatherton Rd overpass both with and without the composite bottom flange is illustrated below.
In conventional “single composite” design, the compact section for larger span girders in the negative moment region can often not be achieved due to limitations on plate thickness.

If the section over the pier was designed as non-compact, local buckling of the web or compression flange prevents the development of the full plastic moment. In this case, AS5100.6 requires that at the strength limit state, the section moment capacity be calculated based on the elastic distribution of stress, with the stresses calculated based on the build up during staged construction.

The diagram below plots the applied bending envelope as calculated for the Heatherton Rd Overpass. Also plotted is the section capacity based on the compact double composite section, the compact single composite section, and the single composite non-compact section.

*Figure 2.1 – Plastic Section Capacity Diagrams*

*Figure 2.2 - $M^*$ vs $\mu$ Profile per Box Girder – Heatherton Rd Overpass*

Legend
(a) Section Capacity – Double Composite - Compact Section
(b) Section Capacity – Compact Section - Steel Only
(c) Section Capacity – Single Composite - Non-Compact Section
3 WEB PLATE DESIGN

The webs for Eastlink’s steel composite box girders were fabricated from 16mm plate up to 2.5m in depth. These were designed as transverse stiffened webs in accordance with the provisions of AS5100.6 Article 5.10. The transverse stiffeners were designed to prevent web buckling under the applied shear, and were proportioned using the code’s minimum area requirements.

The web design was automated using a purpose written Excel spreadsheet for calculating the requirement for transverse web stiffeners. For the continuous bridges the “tension field” at the top of the web from the negative moment provides a significant strength benefit in preventing buckling under the applied shear. This strength enhancing effect is accounted by AS5100.6 using the coefficient $\alpha_d$ in the stiffened web shear buckling capacity equation. This factor alone gave an enhancement against buckling of up to 60% in the peak shear regions at the piers.

A particular design requirement of AS5100.6 Article 3.9 is that “for composite box girder bridges, the total effective thickness of web-to-flange welds shall not be less than the thickness of the web”. To achieve this requirement, two 10mm full penetration fillet welds at the web to flange junction (effective throat thickness 8mm) were required to achieve the total effective thickness of 16mm. On this bases, web plate thickness greater than that adopted would lead to excessive welding requirements to adhere to this specific code provision.

![Shear Force vs Shear Capacity (including torsional effects)](image)

Figure 3.1 - Plot of Applied Web Shear vs Web Shear Capacity

Legend
(a) Web Shear Capacity – Unstiffened Plate
(b) Web Shear Capacity – Stiffened Plate (Transverse Stiffeners at 1.5m and 3.0m centres)
In addition to shear buckling capacity, the webs are also checked for the interaction of shear and bending, and compressive bearing action. The web stresses were derived from a Strand7 finite element model as illustrated. This finite element model was also used for determining a number of other design load effects including bracing loads, flange stresses, fatigue stresses, and deck slab load effects.

![Figure 3.2 - Finite Element Model – Plot of Web Shear Forces](image)

### 4 PIER DIAPHRAGMS

Pier diaphragms were detailed as reinforced concrete cast inside the steel box following girder erection. The function of the diaphragm is to transfer the shear force in the webs to the bearings that support each of the girders. The transfer of the web shear to the in situ concrete diaphragm was achieved using welded stud shear connectors to the web plates detailed in accordance with the provisions of AS5100.6 Article 6.6.

For both bridges, concrete diaphragms cast inside the box girders were provided. The diaphragms were detailed with an access void which was necessary for both maintenance and access ventilation purposed. The typical details adopted for the internal diaphragms including the provision for access are illustrated adjacent.

External steel bracing was also provided between the girders at the piers only. This was necessary for girder torsion restraint.

![Figure 4.1 - Concrete Diaphragm Detail](image)
Another design issue was that the erection staging required that the girder mass be supported by the steelwork alone prior to the diaphragms being cast. This was achieved using welded plate stiffeners over the bearing plates with adequate strength to carry the steelwork self weight and the mass of wet concrete during the casting of the permanent diaphragm. A finite element model was used to determine the local stresses in the temporary diaphragm during this construction stage.

The stresses induced by the girder bearing support at the pier prior to casting of the permanent concrete diaphragms required careful consideration. Stiffener plates were detailed to transmit the bearing reaction loads to the webs and transverse diaphragm plates.

A finite element model was again used to determine the stresses at this stage of construction. Both an elastic and non-linear plastic analysis was adopted to determine the adequacy of the proposed temporary stiffener arrangement.

Based on an elastic analysis, peak tensile stresses in the stiffeners that exceeded the yield stress existed – although it was recognized that minor post-yield deformations would redistribute these stresses through the alternate load paths. For this reason a non-linear plastic analysis was run to determine the magnitude of deflection that would occur beyond the onset of local tensile stress yielding. Based on this analysis, the deflection of the stiffener plate system to achieve the applied load was proven to be negligible (<1mm).

5 FATIGUE

Analysis for fatigue effects is an important design consideration for steel composite box girders. The following considerations are important when considering fatigue:

- Number of traffic counts (in particular heavy commercial vehicles) contributing to the fatigue stress cycles as defined in AS5100.2;
- The stress range in the steel under the fatigue vehicle loading;
- Fatigue Detail “Category Classification” as defined in AS5100.6; and
- Development of suitable fatigue details if necessary to achieve acceptable fatigue stress ranges.

The more critical fatigue details were encountered on the Princes Highway Bridges which had significantly higher commercial vehicle traffic counts than the Heatherton Rd Overpass. Critical fatigue details that required particular consideration were the tension flanges particularly in the location of any transverse welds, at the bottom of the welded transverse stiffeners where the webs carry tension stresses, and for the design of shear studs. To avoid transverse welds in critical tension flanges, the detail illustrated was developed – this allowed a bolted connection in lieu of a transverse weld to the flange which allowed a significant improvement in the fatigue detail category in accordance with AS5100.6.

![Fatigue Detail - Bottom Flange Connection](image)

**Figure 5.1 - Fatigue Detail - Bottom Flange Connection**

<table>
<thead>
<tr>
<th>Fatigue Stress Summary - Heatherton Rd Overpass Bottom Flange (φ = 1.0)</th>
</tr>
</thead>
</table>

![Fatigue Stress Summary](image)

**Figure 5.2 – Plot of Bottom Flange Fatigue Stresses**

6 LIVE LOAD ANALYSIS

Live load analysis for the Princes Highway Interchange structures was undertaken using the AS5100 Bridge Code automated loading methods using SAM Integrated Bridge software. One issue of particular note in the live load analysis was the difference in the interpretation of the carriageway definition between the Austroads 92 Bridge Code, and the AS5100 Bridge Code for structures with footpaths. While the Austroads 92 code allowed the carriageway to be considered as the width
between kerbs, the AS51000 bridge code now requires that the width also be considered between traffic barriers (i.e. extending over the footpath). The two clauses are summarized in the table below:

<table>
<thead>
<tr>
<th>Austroads 92 vs AS5100 Code Comparison – Carriageway Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Austroads 92 Bridge Code Article 2.3.5</strong></td>
</tr>
<tr>
<td>“The number of standard design lanes shall be ( n = \frac{b}{3.1} ) where ( b ) = carriageway width between kerbs or traffic barriers (whichever is less)”</td>
</tr>
</tbody>
</table>

This subtle difference between the two codes was interpreted to mean that the Heatherton Rd Overpass, which has a footpath without separation by a traffic barrier, needed to be checked for two cases:

- **Load Case 1**: Carriageway between footpath kerbs plus footpath concrete mass and pedestrian load where applicable;

- **Load Case 2**: Carriageway between outside barriers without footpath concrete mass and pedestrian loads (AS5100 implies in this case that future widening with footpath removal needs to be considered unless specified otherwise).

The latter load case has a significant impact on the load effects generated in the outside girder webs. A typical SM1600 vehicle live load pattern is illustrated below.

*Figure 6.1 - AS5100 Live Load Automation*
7 ERECTION STAGE ANALYSIS

The erection stage analysis was an important consideration for determining the serviceability limit state stresses. As the sections were detailed as compact, the stage analysis was not necessary for flexural analysis at the ultimate limit state (refer Section 2 above). However, the stage analysis for the girder erection procedure was particularly important for construction. The staging required that the girder segment support levels at temporary support towers and piers be adjusted to control the rotations at the girder splice locations. The rotations of adjacent segments at the splice location needed to match such that the bolted splices could be assembled. The erection staging developed for the Heatherton Rd Overpass is illustrated below.

8 CONCLUSION

Steel composite box girders developed for Melbourne’s Eastlink Project were an efficient solution for achieving spans up to 65m. The method of analysis and details developed are considered to have provided further efficiencies allowing steel to have been the viable choice over the concrete alternatives.

9 REFERENCES

1. CISC, “Steel Bridges Design – Fabrication – Construction”, Canadian Institute of Steel Construction 1994
2. STROH and SEN, “Steel Bridges with Double Composite Action”, Transportation Research Record No. 1696