Introduction

The Eastlink Exit Ramp to Ringwood Bypass Bridge forms a part of the 45 Km long Eastlink linking Mitcham and Frankston, approximately 20 km to the east of Melbourne.

The concession to operate Eastlink for a 34 year period was won by Connect East. Connect East appointed Thiess John Holland Joint Venture to design and build the project. CW-DC, a wholly owned subsidiary of Connell Wagner provided design services for northern, Mitcham and Ringwood, region of the project which commence at the end of Eastern Freeway at Springvale Road and extends to the Ringwood region.

The exit Ramp to Ringwood Bypass Bridge is located in the northern region of the project. The bridge will carry the Eastlink exit ramp traffic (from the northbound main carriageway) east to the Ringwood Bypass and will form the top level of a three level interchange at about 18m above existing ground level. The second level of the interchange consists of two adjacent bridges, which will carry the Eastlink main carriageway (one for each direction) over the Mullum Mullum Creek. The third and the lowest level of the interchange is within a cut and is the Melbourne bound Eastlink entry ramp from the Ringwood Bypass. The arrangement of these structures and routes are shown in Figure 1.

The superstructure of the exit ramp bridge comprises of twin steel trough girders made doubly composite with concrete deck slab and the concrete bottom flange slab as shown in Figure 2. The bridge is 11m wide between the kerbs and is continuous over 6 spans with an internal concrete diaphragm providing the continuity at the piers. The alignment is on a 230m horizontal radius and the total length of the bridge is also 230m as shown in Figure 3.

All piers except one consist of a single reinforced concrete column and a cantilever crosshead. Piers are on average 16m high and due to their slenderness the top of piers 3, 4 & 5 were fixed to the deck through single fixed pot bearings under each of the girders. Pier 5 consists of twin reinforced concrete columns and a crosshead. Bored piles support the piers and abutments. The expansion joints for the bridge are provided at the two abutments.
Design Parameters

The bridge carries two lanes of traffic east to the Ringwood Bypass. The width of the bridge between kerbs is 11m due to the need to comply with sight distance requirements for vehicles travelling at 80 Km/h. The road design was completed in accordance with VicRoads Road Design Guidelines.

The bridge design was in accordance with AS5100 and AASHTO design guidelines for horizontally curved steel girder highway bridges. AASHTO was extensively referenced to compliment the design rules given in AS 5100 and this paper highlights the relevant aspects of AASHTO requirements that were considered in the design.

Bridge Superstructure

The bridge superstructure is comprised of twin steel trough girders with a concrete deck consisting of precast Transfloor slab formwork and in situ overlay. The top deck acts compositely with the steel trough girders in a conventional manner for superimposed dead and live loads. As such, the design of this aspect of the superstructure is not discussed in this paper.

The following sections discuss some of the innovative features introduced in the superstructure design to produce a cost-effective structure that can be easily built.

Proposed Construction Sequence of the Superstructure

The construction sequence of the superstructure will typically commence with full span girders individually lifted into position and landed onto temporary bearings placed either side of the permanent bearings on the crossheads. Each girder is also temporarily supported against torsion due to the curvature of the alignment and incidental wind loading.

Above each abutment, and pier numbers 1 to 3, external K bracing between the pairs of girders will be installed at each end of each span immediately following the temporary propping stage. The external K braces are aligned with the internal steel diaphragms located 0.75 metres from each girder end at these locations. Above the heavily skewed piers, numbers 4 and 5, an alternative method for girder interconnection was required, and this is discussed later in the paper. Girder erection will commence with span 2, then span 1 followed by spans 3, 4, 5 & 6. The Contractor due to access requirement proposed this sequence for their crane.

Following the installation of the permanent interconnecting K bracing at all intended locations, the internal reinforced concrete diaphragms and the bottom slab will be cast. External reinforced concrete diaphragms above piers 4 and 5 will also be cast at this stage. After all diaphragm concrete has gained strength, the temporary bearings will be removed and loads transferred to the permanent single bearing. Precast concrete formwork will then be placed on top of the girders starting at the north abutment of span 1 and progress to the south abutment of span 6. The in situ deck will then be cast for the entire bridge.
Following completion of the in situ deck all other bridge elements such as parapets, asphaltic concrete, linemarking, expansion joints and underbridge lighting, will be installed.

This construction sequence was modelled in the software, which has the capability to manage these sequences. All the stage stresses were accumulated automatically in the program and considered in the final design.

**Design and Detailing of Steel trough Girders**

Steel trough girders are usually subject to warping and distortion of the sections. In this bridge such actions are further accentuated by the fact that the bridge is curved and two piers are heavily skewed. Internal cross bracings at close intervals were therefore introduced to prevent or minimise the distortion.

AS 5100 does not provide any guidelines for design of internal cross bracing against distortion, however AASHTO and BS 5400 Part 5 do provide guidance. The design of the bracing was carried out according to AASHTO by providing adequate stiffness via the bracing to prevent distortion.

The steel trough girders carry self-weight and the loads from precast and in situ concrete slabs. Prior to establishment of composite action with the concrete deck, the top flange will also have to be restrained with adequate lateral bracing to prevent buckling. The lateral bracing also aids in the torsional resistance of the girder prior to composite action.

The trough girders are subject to significant torsion in association with bending due to their curvature and skewed spans and are subject to co-existing lateral bending due to wind loading and fixity at piers. AS 5110 Part 6 Section 5 which covers generally the design of beams does not address torsion explicitly. The design of these girders was therefore carried out in accordance with AASHTO as this provides clear guidelines.

At the skewed pier locations (pier numbers 4 and 5) the steel girders in plan view could not be terminated with a square end as used at the remaining piers and abutments. Instead, the girder ends were aligned with the centre line of the skewed crossheads. Whilst this added complexity to the design at these ends during construction stage and increased the degree of difficulty in fabrication of the ends, it also provided significant construction benefits. The skewed end arrangement meant that temporary girder supports that are positioned directly beneath the internal steel diaphragms could be supported off the permanent crosshead without the need to provide temporary cantilevered support beams mounted on the sides of the crossheads. In addition, both sides of the girders top flanges could be aligned above the crosshead, which would not have been the case if the girders were made square ended. This reduced the complexity of temporary bracing of girders.
Double composite

An interesting concept introduced in the design of the bridge was the use of double composite action over piers. The introduction of a bottom concrete slab led to a reduction in the thickness of the bottom steel plate and eliminated the need for longitudinal bottom flange stiffeners.

The bottom concrete slab’s other purpose was to provide a transfer mechanism for the compression forces acting on the bottom steel flanges of the girders on either side of bearings. The bottom slab extends away from the diaphragm along the girder for a length proportional to the length of the span. Typically this was 6m. The slab was reinforced longitudinally and transversely and made composite with the bottom flange using shear studs welded on a 150mm square grid.

Shear studs are also located along the webs near the bottom flange, which is subsequently encased in concrete to form the composite bottom flange. This provided additional benefits by preventing the web plate buckling within this area and thereby reduced the web slenderness.

AS 5100 currently does not mention doubly compact sections. AASHTO design guidelines were again referenced as they explicitly discuss doubly composite actions and offer guidelines for design of these elements. As such the doubly composite design was carried out in compliance with AASHTO.

Concrete Diaphragms

Another interesting feature of the design was the use of internal concrete diaphragms within the troughs at piers and abutments. The concrete diaphragms were used to join the full span length girders directly above the bearings. The adoption of the concrete diaphragms within the girders means that steel sections of the adjacent girders are not required to be joined by bolting or welding. Webs and flanges of the adjacent girders were extended to finish 40mm from each other to provide the formwork for the construction of the internal concrete diaphragms.

The top flanges of the adjacent girders are however joined at the pier locations due to Contractor’s preferred construction sequence. Bolting of the top flange was the only means to provide the continuity of the girder, required to resist actions due to the weight of the concrete deck slab, prior to its curing. Another benefit of the continuity of the top flange is that the demand for the longitudinal reinforcement in the deck slab was reduced significantly, eliminating reinforcement congestion on the curved deck.

The bottom slab and the internal concrete diaphragm therefore eliminated the need to directly join the bottom flange and web steel plates, which is always a difficult site operation and would have required heavy steel works, with either significant onsite welding or high strength bolting on all webs and flanges.

The loads from the steel girders are transferred to the diaphragm via shear studs located on the internal faces of the webs and which is then transferred to the permanent bearings.
Due to the complexity of the behaviour of the diaphragms many different 3D models were used to understand their behaviour. These 3D analyses were also complimented by Finite Element analysis of the skewed spans to ensure that robust connections are provided at this complex interface.

**Permanent External Girder Bracing**

The permanent external steel “K” bracing provided at each span end temporarily stabilises the steel girders during construction and provides torsional restraint to the structure during operational conditions.

The “K” bracing provided at abutments and crossheads are radially aligned to the curved bridge. However, for the two heavily skewed crossheads at piers 4 and 5 an alternative bracing mechanism was adopted.

This consisted of a large reinforced concrete section 1.8m deep x 1.5m wide, which was located directly above and aligned with each of the skewed crossheads. This is connected to the internal reinforced concrete diaphragms via reinforcement positioned through holes in the webs.

The external concrete diaphragms will be formed with temporary steel plates. The exposed concrete diaphragm will be painted the same colour as the steel girders so as to provide a continuity of appearance of the steel superstructure.

**Transfloor deck panels**

Another major design feature that will simplify construction and save time is the adoption of 2.4m wide Transfloor precast deck formwork panels, which span the full width of the bridge deck.

The “one piece” precast Transfloor panels, will each be made up of five concrete segments connected with embedded trusses that run continuously for the full width of the deck. These panels are not rectangular in plan view and their long (11.6m) sides are not parallel as shown in Figure 4. The long sides are radially aligned to match the constant curved alignment of the bridge. The six trusses within each Transfloor section are, for ease of Transfloor fabrication, embedded parallel to each other. A further advantage of this truss arrangement is that it led to simplified stud spacing across all four flanges within the Transfloor panel. The spacing of studs between the panels however is varied on each flange to account for the tapering of the precast segments.

This design arrangement is critical to eliminate clashes between trusses and studs as these are intended to be preinstalled prior to erection of the precast panels.
Bridge Substructure

Piers

The design of the piers on this bridge was complex due to many different actions that the piers had to resist because of its location between the mid level bridges below it and also its location in the Mullum Mullum Creek flood plain. All piers except for one consist of a single reinforced concrete column and cantilevered crosshead. Two of the columns are heavily skewed.

The main pier (pier 4) is one of the two that is heavily skewed. It is a single column pier that extends from ground level and passes between the two (mid level) bridges that form the main carriageway for EastLink as shown in Figure 1. The pier column is 1.5m wide at the pilecap level and as the central median gap is only 1.25 metre wide between the two mid level bridges, the pier column needed to be reduced in width at some point below the median gap. Moreover, the face of the pier had to be setback beyond the vehicle sway envelope at the EastLink carriageway level. Stepping of the column width (from 1.5 m to 0.9 m), is positioned at mid height of the super Ts used in the midlevel bridge decks so as to effectively hide from view and consequently produce an architecturally pleasing solution.

Although this pier is stepped down to be clear of the vehicle sway envelope at EastLink level, the design of the pier still considered the full impact load as per AS 5100.

In addition, all piers are located within the flood plain of the Mullum Mullum Creek. As a consequence all piers had also to be designed for 2000 year return flood loading.

Piers 2, 3 and 4 are about 18m high. Due to the constraints mentioned above Pier 4 has a relatively small width for this height of column. It was therefore not possible to design these columns as free standing cantilever columns. Accordingly another novel solution was adopted. Fixed pot bearings were introduced at the top of piers 3, 4 & 5 to enable the column and the deck to act as a frame. The locking of the deck with the piers attracted additional forces on the piers due to creep, shrinkage and temperature effects of the deck. Nonetheless this structural system made it possible to design Pier 4 & 5 with a 0.9 m and 1.5 m width respectively. The locking of the deck for the two spans however provided an advantage in terms of the resistance to breaking loads, as all three piers were able to share the loads relative to their stiffness.

During the analysis, it was initially thought that it may also be preferable to fix the bearings on top of pier 2 so as to reduce the effective length of the column and thereby optimise the column section size. However fixing the bearings in translation lead to large bending moments at the pier base and piles. By releasing the bearings the moments were significantly reduced however the pier had to be made bigger in cross section to account for slenderness limits. Increasing the pier dimensions was preferred in this case rather than fixing the pier at the top and increasing the pile sizes.
Crossheads

Due to the safe stopping distance sightline requirements of the lowest level on-ramp, pier 4 could not be positioned directly below the upper level deck centre line as can be seen from Figure 1. The column offset and the high skew of the pier meant that crosshead was unsymmetrical and had a large cantilever. The depth of this crosshead had to be restricted to maintain a 5.4m height clearance under the crosshead for the EastLink carriageway and as a consequence this crosshead has been designed as a prestressed concrete element. All other crossheads are designed as reinforced concrete members.

Bearings

Each of the trough girders sits on a centrally placed single bearing. The movement of the bearings was determined from analysis of creep, shrinkage and temperature effects of the deck as well as their interactions with the relative stiffnesses of the piers. Pier 5 consists of two columns and a crosshead. Due to the high skew and relative short height of the pier columns it was the stiffest section of the bridge support system and so acted as an anchor point for the horizontal effects of the superstructure.

Typically on curved bridge structures all bearings are orientated toward a central fixed point. On this bridge at all the free piers and abutments, one bearing was aligned tangentially to the radii of the girder and other bearing was free to move in any direction. Whilst this arrangement added bending and shear loads to the pier columns, this simplified the girder fabrication by allowing square cut outs to be adopted in the bottom flanges of the girders around the bearings in the heavily skewed piers. This also simplified the setting out of the bearing directions in all the free piers and in the setting out the shear studs on the bottom flange.

Piles & Pilecaps

The reinforced concrete bored piles varying from 750mm to 1000mm diameter, were selected due to the ground conditions and these piles were modelled with soil springs in a conventional manner to design to resist the axial compression, tension and the bending moments acting on them.

A 1.5m deep pilecap was designed using strut and tie methods and the reinforcement detailed to resist all forces acting on them.

Maintenance

The design has also provided for two maintenance accesses to the internal sections of the girders to carry out regular inspections. A three coat paint system consisting of micaceous oxide, epoxy and polyurethane has also been proposed for all the steel elements of the bridge. Total thickness of the paint system is 225 microns.
Conclusion

The various innovative solutions incorporated in the design of the Eastlink Exit Ramp to Ringwood Bypass Bridge have produced a cost effective solution, which will be easily built within a short time frame.
Fig 4