PEDESTRIAN BRIDGES ON THE REGIONAL RAIL LINK PROJECT, VICTORIA

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ABSTRACT

The Regional Rail Link (RRL) Project involves the construction of new regional tracks from Southern Cross Station in Melbourne, through to Sunshine then west to Werribee. Within the Footscray to Deer Park section of the project, major pedestrian bridge works were undertaken at Nicholson Street and HV McKay Reserve to replace existing bridges by new structure spanning across the enlarged rail corridor. This section of the works was delivered by an alliance consisting of Balfour Beatty, Metro Trains Melbourne, Parsons Brinckerhoff, the Regional Rail Link Authority, Sinclair Knight Merz, Thiess and V/Line. The architectural input for the design was provided by Hassell Studio.

This paper focuses on the special design features of these two pedestrian bridges in the brownfield environment, including the impact of installation in an operating rail corridor, limitations in space and operating conditions imposed due to the location of the bridges, aesthetically pleasing design and the dynamic analysis involved in the design.

INTRODUCTION

For the RRL project, two pedestrian links over the existing metro and regional tracks were required to be demolished and rebuilt to allow for the widening of the corridor. The bridges are pinpointed on the map in Figure 1 below, at number 2 and 7.

The HV McKay Reserve Bridge is a historically-listed link between the HV McKay Memorial gardens and Harvester Road in Sunshine town centre. It was built by Hugh Victor McKay circa 1920 between the Harvester Housing Estate and the Harvester Factory. HV McKay wanted to facilitate the access to his factory from the Sunshine Town Centre. The old bridge consists of multiple spans built from a mixture of timber and steel material. A new bridge was required to span over the enlarged corridor and to comply with Australian Standard AS5100 Bridge Design.

The new bridge is a 66 m single span 3.8 m deep Warren Truss structure which minimises the structural depth and avoid locating piers immediately adjacent to the rail tracks. The bridge provides full access to people with disability with lift access from ground level. The bridge screens incorporate a representation of the HV McKay gate symbol to remind users of the bridge history.

The Nicholson Street Bridge in Footscray links the town centre to Victoria University main campus and to the main road access to Melbourne-Footscray Road. It is also an important bus route. The old bridge was a two span concrete bridge carrying road and pedestrian traffic reinforced with rail sleepers. In order to allow for services to be relocated ahead of time a separate pedestrian bridge was built first, fitted with 34 conduits within its structural depth. The tight geometric constraints to maintain both the current road and rail levels only allowed for a structural depth of 1.1 m to span the 33.5 m wide rail corridor. A composite steel and concrete structure was designed to provide the required strength and allow for the conduits to be fitted between the steel girders. Anti-throw screens were provided to protect the public from the rail corridor below.
HV MCKAY RESERVE BRIDGE

To minimise the number of vertical members and maximise the visibility from the inside of the bridge, the truss design is a square Warren Truss type which uses square hollow sections for both the chords and the braces. High strength steel with a yield strength greater than 450 MPa was specified to minimise the structural depth for this large span. The concrete deck does not contribute to the strength of the truss but provides a continuous walking surface. To minimise the height of the abutment piers the truss is designed with a pronounced camber.

As shown in the general arrangement in Figure 2 above, the diagonal brace members are out-of-phase, one side to the other. This was advantageous to disguise the fact that the abutments were not parallel, and thus the north and south truss were different length.

The construction of the truss was as follows:

- Fabrication, assembly and painting of the truss by Haywards in Tasmania of three 22 m long segments.
- Delivery of the segments on site, and assembly of the segments with bolted splices.
• Erection of the truss in a single lift over the existing rail corridor, using three cranes and midair transfer.
• Casting of the bridge deck over Bondek sheeting.

Figure 3: 3D modelling of the steel truss

The requirement to eliminate site welding introduced bolted splices for both the chords and diagonal braces, and special attention was given to keep them as aesthetically pleasing as possible.

Furthermore the truss design followed the design guidelines of the Comité International pour le Développement et l’Etude de la Construction Tubulaire (CIDECT) manual [1], considered internationally to reflect the ‘state of the art’ in steel hollow section joint design practices.

Figure 4: Erection of the truss

Truss member joint design

The joint behaviour of hollow-section gantry trusses has to be considered at the early stages of the member design. In comparison to the rigid moment connections used in portal frame warehouses – which typically consist of I-beams and stiffened endplates – the connections between hollow sections are generally flexible. This flexibility gives rise to the following:

- Large stress concentrations – which can magnify the stresses calculated on the basis of $\frac{P}{A} \pm \frac{M}{Z}$ by up to five times. Stress concentrations are affected by truss and hollow-section
geometries, the state of loading, metallurgical effects, residual stresses, the welding process and geometry of welds, and any post-weld improvements.

- Modes of failure which are not otherwise a problem with heavily-stiffened moment connections. For example hollow-section trusses may be subject to – “chord plastification”, “chord punching”, or “chord buckling”.

Hollow-section connections are generally identified as “T”, “L”, “X”, “N”, and “Y” connections, with some further variation or combination. Connections may be “gap”, or “overlap” connections. The former may be easier to weld, however are most often weaker, and more prone to fatigue, than overlap connections; overlap connections are generally stiffer.

It is incorrect to design the connections in hollow-section trusses on the basis of standard beam theory, i.e. \( \frac{P}{A} + \frac{M}{Z} \). The designer requires a method which recognises the concentrations of stress at various points around the perimeter of each connection, as well as the possible modes of failure.

**Design principles**

One of the key principles to optimise the design of square hollow section joints in trusses is to use thick-wall chords and thin-wall braces. The stiffer walls in the chord resist loads from the brace members more effectively, and the joint resistance thereby increases as the width-to-thickness ratio decreases. The thin walls in the brace require smaller welds as a result. Due to the high strength steel used in the design, a reduction factor of 0.9 to the joint capacity is required by the CIDECT manual.

For aesthetics, 45 degrees braces were preferred. To simplify fabrication all joints were designed as K gap joints, as per Figure 5 below, since they are easier to prepare, fit and weld. For this type of joint, a deflection allowance of 15% of the total calculated truss deflection should be used to account for the flexibility of the joint.

The fabricator was given the liberty to choose between fillet and complete penetration butt weld to achieve the most economical outcome. Only the minimum required weld throat thickness was stipulated on the drawings.

![Figure 5: Typical K gap joint](image)

**Bolted splices design**

The design of the splices needed to be as visually unobtrusive as possible and work in both tension and compression. Typical bolted chord splices include flange-plate joint with bolts on all sides of the hollow section. This splice type was however rejected due to its poor visual aspect.
Alternative options were considered, such as bolts going through the section and resisting the force in shear. The bolts however cannot be fully tensioned without the addition of compression tubes inside the spliced section, as they would crush the walls. Refer to Figure 7 below. Due to the large number of bolts required this option was not practical.
An innovative solution was developed to “hide” the bolts and maintain the profile of the chord. A splice plate was introduced inside the hollow section on all sides with the adequate number of bolts on each face of the section to resist the design compression or tension force. Because the bolt nuts will not be physically accessible for tensioning of the bolt when the splice is closed, the holes in the splice plate were tapped to replace the nut.

![Figure 9: Chord splice before assembly](image)

Interior splice plates on all four faces of the splice, used as a ‘blind nut’, were made of high strength Bisalloy steel. BISPLATE® 80 typically achieves tensile strength of 800 MPa which is equivalent to the tensile strength of a high strength nut, and is of similar thickness to have an equivalent thread length. The high strength steel also allows the tension force to be transmitted through the plates even after taking into account the bolt holes reduction.

The bolts are fully tensioned grade 8.8/TF. The faying surfaces are painted with inorganic zinc to provide a reliable friction coefficient and prevent slip in the serviceability limit state.

![Figure 10: Chord splice after assembly](image)

**Brace splice (webs)**

Similarly, the brace splices design was driven by the desire to achieve minimal visual disturbance. The design consists of a splice plate welded to two walls of the Square Hollow Section (SHS) brace member, bolted to a mirror plate on the other end of the brace. To prevent shear lag failure, the welded length to the SHS member must be maximised. This can be achieved by providing slots through the walls to fit the splice plate. Once the connection has been bolted up, the joint is finished by additional non-structural cover plates in the shape of the brace member.
Figure 11: Brace splice detail

Figure 12: Chord and brace splices before cover plates are installed
NICHOLSON STREET PEDESTRIAN AND SERVICES BRIDGE

The structure is made of 925 mm deep steel I girders composite with a 200 mm thick concrete deck, made of the precast layer (Transfloor) and an in situ topping. In plan, one edge of the bridge is straight and this other follows a 39 m radius, with a minimum trafficable width at mid-span of 4 m. During construction and before the composite action, the steel girders are stabilised and prevented from lateral-torsional buckling with the U-frame action of Parallel Flange Channel (PFC) member welded to the web stiffeners. Composite action is achieved using shear studs designed in accordance with AS5100.6 Section 6.

Lateral bracing to the bottom flanges was provided to resist rail impact loading to the superstructure as per AS5100.2.
As mentioned previously the brownfield environment constraint imposed a limited structural depth to maintain the existing rail clearance and road level. A composite structure was an economical design which allows for maximum strength with minimum depth required. Nevertheless the critical aspects of this bridge design include:

- Stability of the girders during construction and before composite action. The effective length for bending was calculated as per AS5100.6 Clause 5.6.3. To maximise the girders depth the concrete deck thickness was minimised to 200 mm, the minimum required to achieve the reinforcement cover.

- By nature and due to the large span-to-depth ratio (approx. 30) the bridge is relatively flexible. The dynamic behaviour analysis revealed the first natural vertical frequency of the bridge to be 2.2 Hz which is in the critical range of resonant frequencies of 1.5 Hz to 3.5 Hz provided in AS5100.2 to be investigated as likely to be excited by pedestrian traffic. A transient dynamic analysis was therefore developed to calculate the theoretical dynamic deflections from a pedestrian movement across the bridge.
Dynamic behaviour and site testing

AS5100 describes the pedestrian load as a 700 N load traversing the bridge at a given range of walking speeds. This behaviour can be modelled using the inbuilt transient dynamic capabilities of Strand7 software.

Modelling and findings

The procedure adopted here follows recommendations by Wheeler [2]. Different walking speeds can typically be linked to a corresponding stride length, and therefore an applied vertical frequency in footfalls per second as per Table 1 below from Bachmann and al [3].

<table>
<thead>
<tr>
<th>Walking speed</th>
<th>Forward speed (m/sec)</th>
<th>Stride length (m)</th>
<th>Equivalent vertical frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slow walk</td>
<td>1.1</td>
<td>0.60</td>
<td>1.82</td>
</tr>
<tr>
<td>Normal walk</td>
<td>1.5</td>
<td>0.75</td>
<td>2.00</td>
</tr>
<tr>
<td>Fast walk</td>
<td>2.2</td>
<td>1.00</td>
<td>2.22</td>
</tr>
<tr>
<td>Slow run (jog)</td>
<td>3.3</td>
<td>1.30</td>
<td>2.50</td>
</tr>
</tbody>
</table>

The calculated bridge natural frequency corresponds to a “fast walk”. As suggested by Wheeler, this dynamic loading can be modelled by time-dependant half sinusoidal loading applied at successive locations along the bridge axis, distant from each other by the stride length.

Using Strand7 transient analysis (time-history), the dynamic response can be calculated by assuming a critical damping ratio. Literature review suggested very low value for composite steel-concrete structure. Sétra Technical guide [4] quotes the Comité européen du béton (CEB) information bulletin No. 209 which provides average damping value of 0.6 %. Using this data during modelling it was observed that the maximum dynamic deflection was marginally greater than 1.8 mm, which is marginally greater than the maximum dynamic deflection allowed by AS5100.2.

However, these results do comply with the British and the Eurocode standards, which present acceptability criteria based on acceleration instead of displacement. Besides, it has been shown that the installation of screen mesh fencing (such as the anti-throw screen fitted on the bridge
edge) generally contribute to a significant increase of the damping ratio, because of the friction generated between wires and bolted connections during vibrations and the additional stiffness provided reduces the amplitude of the vibrations. Likewise, the conduits were also likely to modify the response of the bridge.

It was thus proposed to build the bridge as designed, and undertake further testing upon completion of the bridge to determine the “actual” natural vertical frequency and damping coefficient. Provision was made to fix tuned mass dampers to the web of the main girders in the event the measured vibration was deemed not satisfactory.

Testing and outcome

Vibration testing was undertaken on the complete but yet to be opened bridge by SKM Sydney Advanced Analysis and Test group. Three accelerometers were positioned across the bridge deck at mid-span of the bridge so as to measure vertical vibrations due to flexure as well as torsional vibration responses of the bridge at the edges of the deck. The dynamic properties of the bridge were measured using model testing and the maximum vibration response using staged pedestrian walk-by tests.

The first significant finding was that the lowest measured natural frequency of 2.3 Hz closely matched the calculated one. However, the measured critical damping of 1.7 % was notably higher than the damping used in modelling.
After the pedestrian walk by test, the maximum measured vibration level at mid-span of the bridge due to a single 700N pedestrian walk by was approximately 0.8 mm which is less than 50% of the maximum allowable vibration level according to AS5100.2. Even allowing for some test variability, this vibration level is well within the acceptable range. An output of the testing data is presented in Figure 19 below.

Although the vibrations are noticeable to a stationary person on the bridge, the testing concluded the dynamic response of the bridge complied with AS5100.2 and that no tuned mass dampers were required.

CONCLUSION

The new pedestrian bridges built as part of the RRL project have overcome difficult design and construction constraints in a brownfield environment.

For the Nicholson Services and Pedestrian Bridge, the existing conditions required a small structural depth which made the structure flexible and therefore sensitive to vibration. In-depth dynamic analysis and site testing post construction allowed to justify the new bridge compliance with AS5100.

The HV McKay Reserve Bridge link has high heritage significance to the local community. The new bridge structure was carefully detailed to be aesthetically pleasing and satisfies the strength and serviceability requirements of AS5100. The structural design with steel hollow section follows the latest recommendations from international research, by considering the joint design from the beginning, and using proven methods to check the adequacy of the chosen arrangements.

REFERENCES


AUTHOR BIOGRAPHIES

Joakim Dupleix had completed a Master Degree of Engineering in both France and Australia by 2008. He has worked for 5 years with SKM in Melbourne in structural design specialised in bridge design and its associated structures. He has experienced different aspects of the bridge life-cycle: tender to detailed designs and large scale strengthening of existing bridges. Joakim was the Design Package Lead on the HV McKay Reserve Bridge and the Nicholson Street Services Bridge. Early 2014 he moved on to take a new role with VSL Technical Centre Asia in Singapore specialising in the design of construction related equipment for large scale infrastructure projects worldwide.

Bruce Gibbens has almost 20 years’ experience in all aspects of the bridge life-cycle: from concept studies and independent verification, to the load-rating, strengthening, and material-condition assessments of existing bridges, as well as providing technical support during the construction phase. He specialises in segmental box girders, including balanced cantilever design. He has also supervised the design of steel box girders, arches, and integral bridges, as well as complex bridge widenings and railway structures. Bruce once lived and worked in California, enduring the rigors of high-level seismic design. Bruce was Technical Lead for Bridges on Regional Rail Project.
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