 Modification To and Widening Of a Concrete Box Girder Segmental Structure

Richard Lipianin, Principal Bridge Engineer, Parsons Brinckerhoff
Sleiman Mikhael, Senior Bridge Engineer, Parsons Brinckerhoff

1 SYNOPSIS

This paper describes the modifications to and the widening of a precast segmental, box girder viaduct that was erected by the balanced cantilever technique. In broad terms these include alterations to the box girder spans and widening of the viaduct over a length of approximately 790 m as well as new ramps.

Part 1 of the paper describes the alterations to the box girder and the resulting effects on its behaviour. It also describes the demolition strategy and its implementation, as well as widening of the deck in those areas where the widening is supported by the existing girder.

Part 2 of the paper describes the design of steel trough girders used to support the increased deck width and new ramps. This paper describes the philosophy adopted for the widening and some of the details that were developed. It also raises some design issues related to the flange slenderness limits encountered in AS 5100.6—2004 (Ref. 1). The construction staging and erection procedure are explained in relation to the design assumptions and the effect of temperature variation.

2 INTRODUCTION

The West Gate Freeway upgrade project in Melbourne is a significant part of a large project to improve traffic flow on some of Melbourne’s main transport arteries. Currently a significant proportion of the south bound traffic on the Western Link is destined for the south eastern suburbs via West Gate freeway and Southern Link while a similarly large proportion of east bound traffic on West Gate freeway traffic exits the freeway heading north into the CBD. The proximity of the exit and entry ramps creates congestion and traffic hazards as these streams cross at grade. A similar situation exists for west bound traffic. This project will eliminate these weaving problems by grade separating the traffic streams.

Much of the West Gate freeway in this area is carried on two existing viaducts that were constructed in the mid to late 1980s as precast segmental, box girders erected by the balanced cantilever technique. The upgrade works require a considerable amount of alteration to these viaducts. This paper describes the modifications that were carried out. These include the following:

- Extension of the existing cantilever
- Widening by propping the extended cantilever off the existing concrete box girder.
- Demolition of part of the concrete box girder
- Remedial works to partly demolished girder
- Widening and construction of ramps using composite, continuous steel trough girders.
The project is being carried out under an Alliance contracting arrangement set up in early 2007 by VicRoads, the Victorian state road authority. The Alliance partners are Baulderstone, Hyder Consulting, Parsons Brinckerhoff, VicRoads and Thiess.

3 EXISTING STRUCTURE

The existing viaduct was designed in the early 1980s to the NAASRA Bridge Design Specification (Ref. 2). It was designed for T44 and Abnormal Vehicle loading but with an additional requirement that the compression under normal service loads would be not less than 1.5 Mpa (Ref. 3). Any modifications to the original structure as part of this project have to ensure that its existing capacity is maintained. Widening of less than a full lane width (3.5 m) has been designed for the original design live loads (T44) but any widening greater than that has been designed for the live loads specified in AS 5100.2 —2004 Bridge Design Part 2: Design Loads (Ref. 4).

Any narrow widening supported by a beam with spans and depth matching those of the existing viaduct is going to have a different and almost certainly a lower stiffness than that of the concrete box girder. To minimise the effect on the existing viaduct the widening was designed as an independent structure for both serviceability and ultimate loads but its integration with the viaduct was modelled when considering fatigue and horizontal loads.

The viaduct has typical spans of 45 m and a maximum span of 55 m. Figure 1 shows a plan of part of the existing viaduct and some of the works for the West Gate Freeway upgrade.

![Figure 1 – West Gate Freeway Viaduct](image)

4 MODIFICATION TO THE EXISTING BOX GIRDER VIADUCT

4.1 Extension of the Existing Cantilever

The widening on the north side of the east bound viaduct extends for approximately 790 m. It starts at the west end of the northern spine of the existing viaduct (some of which was demolished as described below) but does not extend over its full length so a taper was required at its east end. The cantilever has capacity in excess of that required to cater for loadings in the original arrangement so that some widening can
be achieved without supplementary support. The cantilevers are reinforced with two 20 mm diameter bars at 150 mm nominal centres. One of these bars continues to the end but the other stops 1.5 m from the root of the cantilever. It was found that any extension to the cantilever would be governed by the capacity of the existing structure at the point where these bars are curtailed and that the maximum extension could not exceed 1.4 m.

In the past an extension of this type would have required demolition of the edge of the cantilever to expose a sufficient length of reinforcement to make a lap splice with the reinforcement in the extension. This is a relatively risky and time consuming task when carried out over public roads and other infrastructure so the alternative of drilling holes and grouting bars was considered. It was found that a single layer of N24 bars at 150 mm centres would provide adequate capacity as long as an effective lap splice to the existing reinforcement could be achieved. A total grouted length of 700 mm was determined to be required to achieve the splice taking into account the cover to the end of the cantilever and the fact that the new reinforcement would nominally be located halfway between the existing bars. Typical details are shown in Figure 2.

![Figure 2 – Extension of the Cantilever](image)

Because part of this structure was to be demolished an opportunity existed to carry out trials on the actual structure before a final decision to adopt this approach was made. The tail of the barrier was cut off exposing the edge of the cantilever so that work could proceed without any traffic restrictions on the viaduct. A number of cementitious and epoxy grouts were trialled and all bars were tested in tension to yield. Holes were formed using core cutters. Evaluation of the trials confirmed that this approach was preferable to the demolition option even though considerable time was required to set up for and core each hole.
Once this methodology was accepted for the widening of the cantilever it was adopted for the rest of the project where extension of existing cantilever was to be carried out in one form or another. That meant that on the widening on the north side of the viaduct alone over 5000 holes were required. Fortunately equipment that could drill two holes simultaneously and quickly while located on the cantilever itself was procured which greatly sped up the process.

4.2 Propped Cantilever

There is a limited length within the taper at the east end where the required widening is greater than the capacity of the existing cantilever can cater for but where it was not feasible to install a new girder. The additional widening adds minimal additional load to the concrete box girder so the cantilever was propped to achieve the additional width. Steel circular hollow sections were chosen as struts. These have shear studs welded to the end plate cast into the extension and are bonded to the face of the web of the box girder. Drill-in anchors were used to fix the struts to the box girder while the epoxy cured and these have adequate shear capacity to cater for the design loads should there be a failure in the bond. Struts are space at 2.83 m and are located on the centre line of each segment of the precast box girder. A typical section is shown in Figure 3.

![Figure 3 – Propped Cantilever](image)

4.3 Demolition

To provide the required functionality a new east bound viaduct will cross Montague Street and join the existing viaduct some 250 m east of the latter. It has to be located north of any entry ramp at this interchange. Because of the proximity of the property boundary it could only be located where the original entry ramp had been built which necessitated the demolition of this ramp. One option for the demolition of a structure that has been built as a precast segmental balanced cantilever is to deconstruct it in the reverse order of construction. This option is usually adopted for the same reason that the construction method was chosen originally, that is, to minimise the impact of
the works on the area below the structure. In this area there is unhindered access to the site for construction works so more conventional demolition techniques were thought to be less risky and more cost effective.

Because of the proposed change to the structural arrangement caused by any demolition of this continuous structure a detailed assessment of the existing viaduct had to be carried out. Construction of this spine of the viaduct had started at the abutment where the intended demolition was to be carried out so it was appropriate to start the analysis at this point, some three and three quarter spans to the west of the point where demolition would stop. The girder was capable of cantilevering out to its extent during construction but propping was required when the girder extended beyond that point but was not supported on the previous pier. All the props were to be installed before demolition commenced so it was important to ensure that all the effects experienced during the demolition were considered to ensure that these temporary supports were not overloaded. These included:

- The dead load of the girder
- Differential temperature
- Secondary (parasitic) effects of prestress
- Deflection of the girder during demolition for those supports that were not required to carry load at that stage of demolition.

The propping arrangement during second stage demolition is shown in Figure 4. Demolition was carried out in two stages because only part of the site could be accessed initially.

![Figure 4 – Demolition Sequence](image)

Modification to a Concrete Box Girder, Lipianin 5
4.4 Remedial Works

Demolition of three and three quarters of the west end of this viaduct has changed the configuration of the structure. A detailed re-analysis was carried out to determine the current stress state and what effect these modifications would have on the remaining structure. It was found that the demolition would reduce the dead load hog moment at the next pier by around 50% and increase the maximum sag moment in the next span by around 25%. The new pier constructed to support the new ramps that join this structure provided the opportunity to tie the cantilever down to reinstate the moments that existed before demolition. Figure 5 shows the moments before, during and after these works and Figure 6 shows the tie-down arrangement.

![Figure 5 – Moments during Modifications to the Concrete Box Girder](image)

A total of 10 cantilever and 8 continuity tendons had been cut during demolition. The continuity tendons are not required in what has now become a propped cantilever but prestress at top flange level definitely is. Twelve cantilever tendons remain unaffected by the works and those that have been cut were fully grouted so it is likely that some or all of the force in the tendons will have been retained at some distance from the cut end. A check showed that stresses under service loads at segment joints were within the acceptable range whether the cut tendons had retained their prestress forces or were totally ineffective. Ultimate capacity is adequate under either scenario.

![Figure 6 – Tie-down Arrangement](image)
5 DESIGN OF COMPOSITE TROUGH GIRDERS

5.1 Overview

The total length of composite concrete-steel trough girder superstructures on the West Gate Freeway is about 1650 m. Steel girders were used where the span length was beyond the capacity of the available sizes of the super-T beams. The option of using segmental post tensioned concrete box girder construction was discounted on account of cost and speed of construction in favour of composite construction.

The composite concrete-steel trough girder superstructure consists of multi-span sections between expansion joints designed as continuous over the internal piers. Different steel trough section dimensions were used depending on the maximum span length and width of the new deck. Some spans were located on horizontal curves and required additional consideration of the expected rotation particularly during the construction phase. This part of the paper is only related to the design of new concrete-steel trough girder bridge superstructures attached to the existing viaduct to provide additional width of carriageway. Typical cross sections are presented in Figure 7 below.

![Figure 7 - Typical Cross Sections: (A) Westbound Carriageway Widening; (B) Eastbound Carriageway Widening](image)

5.2 Pier Locations and Span Lengths

Since the new bridge structures abut the existing viaduct, the new piers were positioned with the following in mind:

- New piers are to be as close as possible to the existing piers to minimise the distress in the slab joining the widening and the existing viaduct;
• Where a new pier could not be located on the centre line of the existing adjacent pier (due to existing services or roadways) then the offset should be minimal.

The pier positions were checked on site before the detailed designed progressed. Only minor alterations occurred at later stages and this contributed to the efficiency of the design. The expansion joints of the existing viaduct were extended into the widening. This necessitated the provision of two halving joints across the new widening.

5.3 Selection of Overall Structural Shape and Dimensions

The new superstructure was required to match the existing superstructure depth (2 m) for aesthetic reasons and to maintain the existing vertical clearances. This depth was sufficient for the longest new composite steel girder spans. The deck consists of 70 mm thick precast Transfloor panels and a cast in situ concrete overlay with a uniform thickness of 180 mm so that the total thickness of the concrete deck is 250 mm. Girders were fabricated curved in plan and twisted about their longitudinal axes to match the road geometry. They were also pre-cambered so that after casting a uniform thickness of overlay the finished surface matched the road profile.

The girder webs were sloped to minimise the bottom flange width while providing top flange spacing to suit the deck width.

A minimum top flange width of 400 mm was adopted to accommodate the Transfloor panels and shear studs. Figure 8 shows a typical cross section of the steel trough used for the eastbound widening.

5.4 Top Flange Bracing

The primary role of the top flange bracing is to provide torsional stiffness and prevent spreading of the top flanges due to sloping of the webs during construction. The inclined bracing members are also subject to compression forces when the steel girder is under positive bending action under construction loads.
5.5 Internal Bracing

The primary purpose of the internal bracing is to maintain the shape and control the distortion of the box girder. The effective torsional stiffness of a cross-section is a function of both the distortion and St Venant components. Reducing distortion allows the torsional stiffness of a section to approach that derived from the St Venant constant.

The K-frame is preferred over the two diagonal cross braces since the bigger opening facilitates access during routine inspection and maintenance. One diagonal member was only provided as internal brace for the eastbound widening due to the small width of the steel trough.

The internal bracing frames are located at the intersections of the top flange bracing members. By adopting a given member size and varying the spacing of the frames along a typical girder in a model, the internal frame spacing was selected for each bridge (8 m for the eastbound carriageway widening and 6 m for the westbound carriageway widening between Normanby Road and Clarendon Street).

Transverse web stiffeners were provided at the same locations as the K-frames as they were used to connect the bracing members to the webs. Additional transverse web stiffeners were provided as necessary to satisfy shear strength requirements. Figure 9 below shows typical internal bracing arrangements used on this project.

![Figure 9](image_url)

Figure 9 – Typical Internal Bracings for the Westbound and East Bound Widening

5.6 Structural Analysis

The bridges were modelled using Space Gass. The models included horizontal curvature so that torsion loads produced by the models did not need for any correction.

The transformed section properties were used in the spans apart from 15% of the span each side of the support where the concrete section was neglected but allowance was made for an assumed quantity of reinforcement in the deck.

Special attention was given to the expected differential deflection of the top flanges of the horizontally curved girders during construction as this might have lead to difficulties for deck construction. A finite element model was developed and the difference in vertical deflections for the worst case was found to be in the order of
Modification to a Concrete Box Girder, Lipianin

8 mm. This was considered not sufficiently significant to warrant an external bracing system. Figure 10 below shows the finite elements model and the deflection results.

![Finite Elements Model of a Curved Girder and Its Deflection under Construction Loading](image)

Figure 10 – Finite Elements Model of a Curved Girder and Its Deflection under Construction Loading

5.7 Structural Design

A minimum 12 mm thick web was used. The web thickness was increased to 16 mm or 20 mm at supports as necessary to provide adequate shear capacities with a reasonable spacing of web stiffeners.

The bottom flange thickness at mid-spans ranged from 16 mm to 50 mm and was 25 mm for most of the spans. The bottom flange thickness was generally 32 mm over the supports with a maximum of 60 mm.

Steel diaphragms were provided over the supports. Typical diaphragm plate thickness of 25 mm was used. 750 mm diameter circular openings were provided in the diaphragms to provide access for future inspection and maintenance.

5.8 Fatigue

Girders, their shear studs and splices were checked for fatigue under fatigue loading. It was found that the girder plates and weld details are not critical in most cases since the new section is integral with the existing viaduct allowing load distribution to it. The most critical detail was found to be the welded connection of the transverse plate on top of the diaphragm with the top flanges.

5.9 Design Issues

It was found that the section capacity in the hog region can drop sharply with an increase in the section slenderness, $\lambda_s$ from the section plasticity slenderness limit, $\lambda_{sp}$. The previous design code (ABDC – Ref. 5) provides smooth transition of
calculated section capacity as $\lambda_s$ varies from $\lambda_{sp}$ to the section yield slenderness limit, $\lambda_{sy}$. Table 1 below provides a comparison of section capacities of the steel trough girder shown in Figure 8 whose dimensions are as follows: 400 x 20 mm top flange, 20 mm web, 900 x 50 mm bottom flange, 4000 x 250 mm slab reinforced with 30N16 top and 30N16 bottom, steel grade 350.

Table 1 – Comparison of section capacities in hog based on AS5100.6-2004 and ABDC 1992

<table>
<thead>
<tr>
<th>Parameter</th>
<th>AS5100</th>
<th>ABDC</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section slenderness, $\lambda_s$</td>
<td>102</td>
<td>102</td>
<td>Governed by the web</td>
</tr>
<tr>
<td>Section plastic slenderness limit, $\lambda_{sp}$</td>
<td>97</td>
<td>97</td>
<td></td>
</tr>
<tr>
<td>Section yield slenderness limit, $\lambda_{sy}$</td>
<td>135</td>
<td>130</td>
<td></td>
</tr>
<tr>
<td>Plastic modulus, $Z_c$</td>
<td>9.95E+07</td>
<td>9.95E+07</td>
<td>Z values based on $f_y=340$ MPa</td>
</tr>
<tr>
<td>Elastic modulus, $Z$</td>
<td>7.74E+07</td>
<td>7.74E+07</td>
<td>Governed by the top flange</td>
</tr>
<tr>
<td>Effective modulus, $Z_e$ (mm$^3$)</td>
<td>7.74E+07</td>
<td>9.56E+07</td>
<td></td>
</tr>
<tr>
<td>$\phi M_s$ (KNm)</td>
<td>23700</td>
<td>29300</td>
<td></td>
</tr>
</tbody>
</table>

The figures above show that the bending capacity calculated to ABDC is significantly higher (24% in this example) than when calculated to AS 5100 for sections with $\lambda_{sp} \leq \lambda_s \leq \lambda_{sy}$, especially for those sections where $\lambda_s$ is close to $\lambda_{sp}$.

5.10 Girder Erection

Girders were erected starting from the east end through to the west end of each section of the viaduct. Figure 11 below shows the typical construction sequence adopted for steel trough girder erection and deck construction.

The first girder is erected over the first and second piers with the outer web and bottom flange splice plates attached to the west end using temporary bolts. The next girder is then lifted and placed on the next pier and the splice plates of the previously erected girder at which time the inner web and bottom flange splice plates are installed using a sufficient number of bolts to support the girder weight. The load is then released from the cranes.

Due to the length of each viaduct section (600 – 700 m approximately) and number of spans, any variation in length of a girder due to a site temperature different to that at the time of fabrication would have caused a cumulative error as construction work progressed westwards. This could have resulted in diaphragm locations offset form the bearing centres. The initial plan was to check the diaphragm positions at the time of girder erection and make any necessary adjustment by pre-shearing the elastomeric bearings. However, no significant offsets were recorded on site due to the following reasons:

- Most of the girders were erected during the early part of the night during winter and spring when the temperature of the steel girders was similar to the temperature of the concrete box girders;
• Casting deck overlay stitched to the existing viaduct progressed at almost the same rate as girders erection with a lag of two spans so that the girders were restrained by the existing viaduct soon after they were installed.

6 CONCLUSION

Widening of the existing West Gate Freeway viaduct using steel composite box girders for spans exceeding those within the capacity of the super T prestressed concrete girders, drilling and grouting in bars and the use of struts were efficient solutions for achieving the desired construction speed at a reasonable cost. The structural design together with the details adopted allowed further cost and construction efficiencies.

7 REFERENCES

1. AS 5100.6—2004 Bridge Design Part 6: Steel and composite construction
2. NAASRA Bridge Design Specification 1976