Design and Construction of Bridges for the SH2 Dowse to Petone Upgrade, Wellington

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SYNOPSIS

The NZ Transport Agency (NZTA) is currently constructing a major improvement to the Wellington road system with the upgrade of SH2 in the Dowse to Petone area of the Hutt valley. Beca was awarded the investigation and design of this project in the mid-1990’s and Fletcher Higgins Joint Venture was awarded the main construction contract in 2007.

The project will provide four traffic lanes on an improved alignment with a grade separated interchange and over bridges allowing existing signalised junctions to be removed, and will improve safety and reduce congestion on this important link.

The design and construction solutions for the upgrade not only had to meet the goals of the NZTA, but they also had to be responsive to the natural and man-made constraints that surround the site. Geographically, this section of SH2 is immediately adjacent to the Wellington Fault and shares a narrow strip of land with Hutt Road and the Melling and Wairarapa rail lines.

These challenges have been met by extensive use of precast bridge girders that allow safe and rapid construction of the bridges despite their complex geometry. In addition, careful attention has been paid to programming and sequencing to ensure the works are able to be carried out with minimal disturbance to traffic and the community.

This paper outlines the bridge design and construction solutions and showcases some innovative ways of dealing with the problems that may be encountered on geographically difficult sites in built-up urban environments.

1. INTRODUCTION

The SH2 Dowse to Petone upgrade by the New Zealand Transport Agency (NZTA) is an urban highway improvement project of a scale and complexity not seen in the Wellington region for many years. Encompassing a mix of civil and structural works, the $80 million project includes the construction of seven new bridge structures over a 3km strip of SH2 between the Dowse Drive and Petone areas of the Hutt Valley.

Beca was awarded the investigation and design of this project in the mid 1990’s following strategic studies by the NZTA that exposed the shortcomings of the existing alignment and intersection arrangements to meet growing traffic demands. The location of the project has meant that the design and construction solutions have had to be responsive to the highway as a major traffic link, while being sensitive to the demanding geography of the site.
The main construction contract was awarded to a Fletcher-Higgins Joint Venture (FHJV) in May of 2007 with a scheduled completion date in early 2010. Now eighteen months into the contract, the bridge structures are largely complete and the project has received very positive feedback from the public with regard to traffic management and progress. In light of the severe project constraints, this positive feedback has demonstrated the success of the design and construction solutions.

This paper will describe the development of the bridge solutions with respect to the physical and operational challenges of the site. By using the design and construction of the bridges on the Dowse to Petone Upgrade project as an example, the paper will showcase some innovative ways of dealing with the problems that may be encountered on geographically difficult sites in built-up urban environments.

2. BACKGROUND

The Dowse to Petone section of SH2 in Wellington’s Hutt Valley (see Figure 1) is a vital part of the state highway network that forms the primary route between the Hutt Valley and Wellington City. With average annual daily traffic (AADT) of around 45,000 vehicles per day, this section of highway functions as the main access to the Lower Hutt CBD from SH2 and the western hill suburbs of Korokoro and Maungaraki. In addition, the highway provides access to the Petone rail station, Percy Scenic Reserve and operating businesses in the Korokoro industrial area. The original road layout controlled the access points to and from the highway at three signalized intersections.

The dual function of the highway as an arterial and access road has historically resulted in a clash between local and regional traffic, with motorists from the western hill suburbs having to cross SH2 to get to Lower Hutt city and vice versa. This section of the highway was last upgraded in the early 1970’s, but increasing traffic volumes since this time exposed the inefficiencies in the original road and intersection layout, resulting in worsening congestion and safety problems.
Preliminary investigations into how this section of highway could best be improved began in the early 1980’s. The NZTA objectives for the Dowse to Petone Upgrade were to improve the distribution and access of traffic between the Hutt Valley and SH2 thus addressing delays and reducing the number of accidents along this stretch of the highway. The project also needed to align with Hutt City Council objectives to improve the traffic flow and to create links to the CBD and Hutt Road in order to create a direct economic and social benefit for local businesses and residents.

Strategy and option analysis showed that replacing the at-grade signalized intersections with a grade-separated interchange and over bridges would successfully achieve NZTA objectives, and Beca was appointed as engineering consultant for the Scheme Assessment Works in the mid-1990’s. A number of overbridge and interchange solutions were developed to remove the signalized intersections and allow through traffic on the State Highway to travel unimpeded while maintaining access routes for local traffic.

The final design for the upgrade project included a grade-separated interchange over SH2 at Dowse Drive (Figure 2A), an overbridge to replace the at-grade intersections at Korokoro (Figure 2B), a new access road and associated bridge to service the Korokoro industrial area, an overbridge to access the existing Petone Rail Station car park and an upgrade to the existing Petone footbridge.

Overall, this design required the construction of seven new bridge structures as outlined in Table 1 below. These bridging solutions not only had to meet the NZTA’s immediate objectives, but they also had to be responsive to the natural and man-made constraints that surround the site. Geographically, this section of SH2 follows the path of the Wellington fault (see Figure 1) in a highly urbanized area with operating businesses, the Petone train station, a school, a marae, a scenic reserve, an historic cemetery and private homes. It also shares land with the Melling and Wairarapa rail lines and Hutt road – all of which fit within a 200m strip of land between the Western Escarpment of the Wellington Fault and the primary fault trace itself.

The nature of the site and the importance of the highway as a major arterial also required careful programming and operations during the construction phase of the works in order to ensure minimal disturbance to traffic and the community.
Table 1: Bridge Structures for the Dowse to Petone Upgrade Project

<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dowse Interchange East Bridge</td>
<td>An 85m long bridge that forms the eastern arm of the Dowse Interchange and spans the Melling rail line and Hutt Road.</td>
</tr>
<tr>
<td>Dowse Interchange North and South Bridges</td>
<td>Two near identical 25m bridges that form the raised roundabout of the Dowse Interchange and span both sets of SH2 lanes.</td>
</tr>
<tr>
<td>Korokoro Overbridge</td>
<td>A 60m long overbridge that spans both sets of lanes of SH2 and the Melling and Wairarapa rail lines.</td>
</tr>
<tr>
<td>Park and Ride Overbridge</td>
<td>25m long overbridge that spans both sets of lanes of SH2.</td>
</tr>
<tr>
<td>Korokoro Stream Bridge</td>
<td>15m bridge for the Service Road crossing of the Korokoro Stream.</td>
</tr>
<tr>
<td>Petone Footbridge</td>
<td>Upgrade of the existing steel truss footbridge to span the new highway alignment, the service road and the Melling and Wairarapa rail lines.</td>
</tr>
</tbody>
</table>

3. DESIGN AND DEVELOPMENT OF THE BRIDGE STRUCTURES

Structural Form of the Bridges

The six road bridges on the project employ simple ductile bridge forms with standard precast concrete girders to form the deck spans. Four of these bridges make use of precast, prestressed Super-Tee beams as illustrated in Figure 3 below. These bridges consist of multiple 1200mm or 1000mm deep Super-Tee beams placed side by side with a 200mm minimum thickness in-situ concrete slab topping. The topping slab ties the girders together and forms a diaphragm to transfer horizontal deck loads to the piers and abutments.

These bridges extend between reinforced-earth approach embankments at their ends but these are not relied upon for support. As such, the decks are made integral with the piers and abutments to provide frame action in both the lateral and longitudinal directions. In this way, the Super-Tees are effectively continuous through the crossheads and there are no intermediate movement joints over the length of the bridge which maintains continuity of the bridge deck. The piers and abutments consist of multiple concrete columns over bored piles that extend down to depths of 20-35m until sufficient vertical support is gained through a combination of end bearing and shaft friction.
The final bridge solutions had to be responsive to site specific challenges such as the importance of the State Highway as an essential link for the district and accommodation of the bridge structures within the extensively developed urban environment. The bridge forms chosen had to be economic and buildable, had to look good in their urban setting, and also had to suit the complex cambers and curves required to match the expected speed environment.

Coupled with these requirements, the geological conditions of the Hutt Valley and the location of the adjacent Wellington fault had a significant impact on the design, and the solution had to be very resilient to seismic activity.

Participation by senior bridge engineers from early in the concept stages of the project ensured that bridge forms were realistic and buildable within the context of the site constraints and the severe seismic demands on the bridges. While the overall forms of the highway structures were not new, the way in which they had to be adapted for the constricted site meant that some unique bridging solutions and arrangements were developed.

A number of options for the bridge structures were considered in the early stages of the design including precast U-girders, precast I-girders and in-situ post-tensioned tee girders supported on a variety of pier arrangements. While some in-situ concepts were considered, precast girders were an attractive option because of their ability to be erected and secured quickly over the highway lanes and rail lines. Historically, U and I girders had been used extensively in New Zealand, but a precast girder form that was relatively new, the Super-Tee beam, showed promise that it could to provide
a construction solution that could be manipulated to form the complex geometry of
the bridges and be erected quickly and safely. Using concrete Super-Tee beams was
not a new concept, as the beams had been used in Australia in various forms since
the 1980's; however, the demands of this project tested the flexibility of these
members to the extreme.

The ability of Super-Tee Beams to be cast with shaped and tapered flanges was
essential to form the unusual shape of the Dowse East Bridge which forms the
eastern arm of the Dowse Interchange in Figure 4. The Dowse Interchange structure
is wedged between the foot of the western hills at Dowse Drive on the West, and the
Melling rail line on the East. This resulted in a need for the Dowse East Bridge to
carry the eastern arc of the interchange roundabout over the live rail line, thus
creating its unusual geometry. The bridge required an abutment width at the western
end of 58m which tapers down to 25m at the first pier (see Figure 5). This shape
transitions into a horizontal curve with a variable super-elevation that increases from
0% to 7.5% over its length to match the eastern approach embankment.

Designing the Super-Tee Beams

For the design of the superstructure, detailed 3D grillage models of the bridge span
were set up in the general purpose structural analysis package, Space Gass. The
geometry of the bridge made it difficult to calculate by hand the correct dimensions
for modeling purposes, and so each span was drawn in AutoCAD and exported to
Space Gass as a dxf file. Producing the curved and warped shape of the bridge with
straight girder sections was carried out by exporting the MX civil surfacing model for
the bridge to AutoCAD, laying out the beams beneath, and ensuring that minimum
concrete deck and surfacing thicknesses were achieved accounting for beam tilt and
hogs. Where the super elevation of the deck was high, beams had to be tilted from
plumb in order to minimize the differential deck thickness across the flange width of
the beams. This approach enabled detailed 3D grillage models of the bridge superstructure to be generated for analysis.

The full structural model comprised of longitudinal and transverse grillage members, pier and abutment crossheads, columns, and piles having an effective depth to fixity based on average soil conditions. The spans have been designed to provide continuity across the piers to resist lateral loads, however the bridge decks are assumed to act as simply supported for all gravity loads. As explained by Hambly et al, it is presumed that the construction joints at the beam ends open slightly over time due to creep and shrinkage shortening. The live loads must close up these joints before any hogging moment can be generated. However, shortly after construction the spans will have continuity across the piers. Therefore, the beams were also designed to resist the associated hogging moment at the supports due to live load and superimposed dead load.

Between the seven bridges on the project there were 83 Super-Tee beams to design that were individual in loading and geometry. Due to the number of beams of different lengths, varying properties and demands, a spreadsheet was specifically developed for calculating and checking the serviceability limit state stresses in the girder elements. The analysis results for each of the load cases were exported into the spreadsheet where they were combined to create an envelope of design actions. This spreadsheet also calculated stresses at transfer, stresses at erection of the cast in-situ slab, the long term in service stress conditions as well as the de-bonding requirements for strands at the ends of the girders. The use of this spreadsheet greatly reduced the time needed to design the beams.

4. IMPACT OF THE WELLINGTON FAULT ON THE DESIGN SOLUTIONS

Design Challenges due to the Proximity of the Fault

One of the major design challenges for the bridges on the project was the close proximity of the structures to the Wellington Fault. This challenge was especially pertinent for the Dowse East Bridge. As shown in Figure 6, the main surface trace of the fault passes at right angles and 25 to 50m to the east of the bridge site beneath the Hutt Road approach ramp. Zones of secondary rupture extend over a width between 50 and 110m on either side of the primary fault and beneath the bridge structure itself.

Based on observations of the Wellington Fault scarp, a major rupture of the Wellington Fault occurs every 500 to 700 years. The last surface rupture of this fault is estimated to have occurred around 450 years ago, and so the probability of this fault rupturing within the life of the bridges is relatively high. Ground movements at the location of the fault are expected to be about four to five metres horizontally and one metre vertically. Movement in the secondary fault zone beneath the Dowse East Bridge is estimated at 20% of the primary fault movement (0.8m horizontally and 0.2m vertically) and so the design of this bridge required special consideration to accommodate the potential ground movement in the secondary fault zone. The area of the secondary fault zone could also be subject to liquefaction of the underlying marine sands.
Design of the Bridges for the Fault Effects

The bridge structures on the project have been designed in accordance with the Transit Bridge Manual for a 1000 year return period event. The manual requires that damage is fully repairable following the design earthquake and that a reasonable margin against collapse is provided during a maximum credible earthquake. This includes the effects from direct earthquake shaking, secondary rupture movements and liquefaction.

The bridges were designed using capacity design principles to guard against collapse and to control the amount of damage in the structures by allowing flexural plastic hinging to occur in the columns in a major earthquake. Controlled damage to the columns dissipates energy and is expected to be repairable following the design event. Other elements of the structure (superstructure and piles) are designed for column overstrength to ensure that the plastic yielding is restricted to the columns only, providing predictable seismic behaviour. Structural displacement ductility was limited to 3 but the bridge members were detailed to give higher ductility capacities. This is to allow for some uncertainty in the Wellington Fault movement and liquefaction effects.

Consideration was also given to limiting deck displacements in a lesser level seismic event by providing a sufficiently stiff substructure. Limiting displacements will minimize damage and repair frequency to the adjacent approach embankments, retaining walls and pavements, as well as ensuring stability of the structure. Bridge deck lateral displacement is targeted to be less than 200mm at a full level 1000 year return period earthquake event. This corresponds to a serviceability level (approximately 35 year return period) displacement of 25mm.

As discussed in Section 0, the stiffness of the bridges is provided by designing the bridge piers integral with the deck to form frames in both the longitudinal and transverse directions. The reinforced-earth approach embankments are not relied
upon for support and movement gaps are provided at the ends of the bridge to structurally isolate the bridge from the approach embankments. This has been done deliberately for the following reasons:

- The bridge inertia under earthquake horizontal acceleration exceeds the capacity of the embankment soil acting in passive pressure on the end of the bridge deck by 30 to 50%. This would result in a complex interaction between the bridge superstructure and abutment stiffness that would be difficult to predict.

- The Hutt Road approach ramp on the east sits directly over the primary fault trace and so the behaviour of the ramp under a rupture of the Wellington fault is difficult to predict with certainty. These effects may pull the approach embankment away from the end of the bridge or push the embankment towards the ends of the bridge, imposing high additional loads on the bridge structure.

The effects of the ground movements that occur in the secondary fault zone were assessed by considering the maximum expected movement spread over the length of the bridge by rotation of the bridge deck in plan and vertical movement of the secondary fault wedge. Displacements resulting in a displacement ductility greater than 3 are acceptable but must not exceed the curvature ductility capacity of the hinging elements. Ground beams link the base of the pier columns in the transverse direction to give added robustness in the event of secondary fault movements beneath the bridge.

Under significant earthquake shaking, exceeding a return period of approximately 800 years, the marine sand layer underlying the site could liquefy. To mitigate settlement or collapse in this situation, piles extend to depths below the liquefiable soil layer until sufficient vertical support is gained through a combination of end bearing and shaft friction. Allowance for liquefaction has been made in the bridge design by assuming no lateral support will be provided to the piles by the liquefied material during earthquake loading. Allowance has also been made for the effect of additional vertical loading on piles by removal of vertical support to the non liquefied surface layer above the liquefied material.

**Utilising Mechanically Stabilized Earth Walls**

Given the ground movements expected in a rupture of the Wellington Fault, it was not considered feasible to build a bridge structure within the primary fault trace zone. Consequently, a Mechanically Stabilised Earth (MSE) structure was chosen to form the Hutt Road approach ramp. Shown in Figure 7, MSE walls offer an attractive structural solution that can incorporate texture and relief for urban design. The system consists of compacted earth reinforced with galvanised steel straps that are attached to concrete facing panels. This type of structure is known to be extremely ductile under fault movements and it is expected to be very tolerant to the ground movements of the extent anticipated. While damage to the structure and pavements is expected in a fault rupture beneath the ramp, it is likely that the ramp will remain intact and will be quickly repairable to allow traffic access to the bridges after a significant seismic event.
5. CONSTRUCTION CHALLENGES

Fletcher Higgins Joint Venture was awarded the construction contract for the works in 2007 and the first pile was poured for the Dowse East Bridge in August of that year. This Bridge will be completed in March of 2009 when the Dowse Interchange opens to traffic. The project continues to receive positive feedback from the public, particularly on the project progress and the management of traffic around the works. For such a large project that continues to be under close scrutiny from the community, this feedback has demonstrated the success of the design and construction solutions.

This section describes the challenges that were faced during the construction of the bridge structures and the ways in which these challenges were met.

Piling Works

The Dowse to Petone works involved the placement of 107 bored concrete piles within the site area and over half of these were bridge piles. The interaction between seismic and fluvial processes within the Hutt Valley meant that ground conditions were highly variable and could vary from solid greywacke to bands of round river cobbles and marine sediments within a very short distance. Piling works had to be responsive to the ground conditions as they were encountered, and piles were provided with permanent casing over their upper length in order to minimise the risk of inclusions into the pile concrete. At the same time, the pile design required an uncased length at their base in order to develop friction resistance for vertical loads. Boring polymer was used to help maintain the stability of the uncased length prior to placing the concrete and strict programming attempted to minimise the time between boring the pile and placing the concrete.

The proximity of local businesses and residences to the site also meant that at some of the piles of the Dowse East Bridge, vibration could not be used to install the steel pile casings. Instead cutting teeth were welded onto the bottom of the casings so that they could be bored down rotationally. This process took longer than vibrating the casings down and a larger piling rig was needed in order to develop the torque to overcome friction against the casing; however this method proved suitable for the site and did not produce vibration.
Fabrication and Installation of Super-Tee Beams

The ability to lift an entire span of Super-Tee beams into place during a rail closure or traffic diversion greatly reduced the impact on rail and road access and minimized the effect on local businesses during construction. Brackets to support handrails and deck formwork are able to be attached to the edge girders prior to erection so that the beams create a safe and secure building platform as they are placed. Once the beams are secured and installed with handrails and side protection, road and rail traffic is able to safely pass under the spans while contractors work on the bridge deck.

As discussed in Section 3, the geometry of the bridge structures resulted in very little duplication of beam elements and almost all beams were unique in design and geometry. The lead time for the beams and the design requirement for the beams to be 30 days old prior to erection (to allow strength gain and creep/shrinkage effect to occur) meant that rejection of beams due to non-conformance would have had a major effect on the program, and so meticulous quality control procedures were applied to ensure that beams were correctly fabricated to tight tolerances.

The design requirement that the bridge deck is integral with the piers for continuity meant that extensive propping was required to safely support the girders while the pier crossheads were formed. This propping was designed and fabricated by the Contractor to enable it to be re-used for all of the bridges in a modular fashion. Bearing mounts for the beams were attached to the temporary works frames and fulfilled the dual role of setting the bearing height and horizontal angle of the girder, and securing the girders safely on the temporary supports.

Traffic, Rail and Pedestrian Management

One of the major successes of the works from the perspective of the public and Client has been the management of traffic and pedestrians through the site on SH2 and local roads. The NZTA requirements for the construction contract included provision of sequencing and staging of the works in order to maintain fully operating local road and the State Highway 2 networks. This included carrying out all associated temporary works for traffic and pedestrian management such as temporary road alignments and temporary bridging for pedestrians. The contractor also needed to liaise directly with the rail owner regarding management of works around the rail corridor.

To assist the contactors during the tendering phase, Beca designed a sequence of Traffic Management Plans (TMPs) showing how the work could be sequenced and phased to divert traffic safely and efficiently throughout the site. The Fletcher-Higgins Joint Venture have on the whole used these TMPs to sequence their own work but have also modified the plans to suit their preferred construction sequencing.

Works over the railway lines were required to be undertaken during scheduled rail closures. In the case of works at the Korokoro Overbridge, rail closures were scheduled every 2-3 months (Figure 8). The long period between rail closures meant that disciplined programming was required. All works over the railway line including beam lifts or preparing the existing Korokoro Bridge for demolition needed to coincide with the scheduled closures. Failure to meet these deadlines could potentially delay
the works for weeks. Programming difficulties were mitigated by regular consultation between the Contractor and Ontrack, the track owners and managers.

![Figure 8: Work being undertaken at the Dowse East Bridge and Korokoro Overbridge in close proximity to the rail corridor. (Note the use of temporary steel towers to support the beams until the cross-heads are poured.)](image)

The extent of the temporary road works for traffic diversions was such that the first nine months of the civil works for the project were devoted entirely to constructing temporary pavements to divert traffic away from the bridge construction sites. These works presented challenges of their own and some structures were built in stages when access to the bridge sites was available. For example, the Park and Ride Bridge was built in two halves, four months apart. MSE Walls 5 and 6 continue to be built in stages as traffic diversions and access to the site allows. The Contractor reports that the savings to programme, and the improved safety of sites isolated from the traffic, has far outweighed the additional investment in temporary road alignment works.

6. CONCLUSIONS

To date, the Dowse to Petone Upgrade of State Highway 2 has received very positive feedback from road users, the local council and the community. Despite severe project constraints, the project is on program and within budget. This validates the design and construction solutions, and demonstrates that even on geographically constrained sites, traffic flow can be maintained at full operating capacity with careful programming and traffic management.

The bridge structures have been designed considering the effect of the nearby Wellington Fault, and the bridge forms are expected to be very resilient to ground movements from the Secondary Fault Zone.

Precast, prestressed Super-Tee beams offered a very good solution for the main bridges on the project as they met a number of design and construction challenges including:

- The ability to form bridge spans with complex geometry.
- The ability to be safely and rapidly erected over rail lines, traffic lanes and in close proximity to operating businesses.
The ability to provide a resilient structural solution with beams integral with piers for structures close to the Wellington Fault.

Mechanically Stabilised Earth Walls have been used for the approach embankments as they provide an attractive structure that is very resilient to seismic actions. This type of structure is also expected to be tolerant to the displacements expected from the primary fault trace.

Traffic, rail and pedestrian management has been integral with the design since concept stage and investment by the Contractor in temporary road alignments and traffic management has provided valuable savings to programme and improved safety of the sites within the project.

7. ACKNOWLEDGEMENTS

The authors would like to acknowledge the New Zealand Transport Agency for permission to publish this paper, including Hannah Hyde who continues to provide support as project manager of the Dowse to Petone Upgrade.

They would also like to thank Section Engineers Justin Hall and Eamon Stynes of Fletcher Higgins Joint Venture for their input and Andrew Paterson, Engineer’s Representative for Beca on the SH2 Dowse to Petone site for his ongoing support.

8. REFERENCES