Design and Construction of Dean Street Bridge, Albury

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SYNOPSIS

Sinclair Knight Merz was engaged by Abigroup in 2005 to undertake the road and infrastructure design of the Albury-Wodonga Hume Freeway Project for the Roads and Traffic Authority of NSW. The 17.4km section of road links the Hume Freeway at Wodonga with the existing Hume Highway at Ettamogah. The brief for SKM's Sydney Bridge Group included the detailed design of seventeen road, pedestrian and railway bridges of which one bridge in particular was to be a statement structure for the City of Albury.

This was a 134m long pedestrian bridge which spans across the Great Southern Railway Lines and the new Hume Freeway to link Dean Street with the East Albury Cycleway, thereby linking the Albury CBD with the community on the eastern side of the Freeway.

The bridge has two 67m spans with a central pier between the Rail and Freeway corridors. Given the span lengths of the bridge and the intention to create a landmark structure at the eastern end of the main street of Albury it was agreed with the RTA during the post-tender discussions that the pedestrian bridge should be a cable stayed structure. The design included the provision of:

- a span configuration to accommodate the site topography, foundation materials and design constraints for the Rail and Freeway corridors
- aesthetic qualities consistent with an iconic structure
- close liaison with Abigroup to achieve a detailed erection procedure satisfying all design, constructability, and risk issues
- investigation of the dynamic response of the cable-stayed structure to vertical and lateral pedestrian excitation employing non-linear transient dynamic analyses on Strand7 software in accordance with AS5100.2 guidelines
- a construction sequencing analysis using Strand7, with independent confirmation using Microstran

1. INTRODUCTION

The cities of Albury and Wodonga straddle the NSW – Victorian border. The need for the National Highway, comprising the Hume Highway in NSW and the Hume Freeway in Victoria, to bypass these centres was under active consideration for more than twenty five years. Various route alignments were proposed culminating in the selection of two primary routes in 1995 – an external rural route to the west, and an internal urban route
to the east of the Albury CBD, on the eastern side of the Sydney – Melbourne railway corridor.

These alignments were reviewed in greater detail and ultimately, in conjunction with extensive community consultation, the inner route was selected. In order to minimise the separation of East Albury from the city centre, the approved concept included the provision of numerous access crossings of the Freeway corridor for pedestrians and vehicles, extensive urban design treatments along the Freeway and the development of parklands in East and South Albury.

One of these access crossings is a pedestrian bridge over the Freeway and the Railway corridor at Dean Street.

2. BRIDGE SITE

It was recognised during the planning stages that the pedestrian bridge at Dean Street should be an iconic structure. Dean Street is the main street of Albury, the commercial and civic heart of the city, and its axis aligns to the west with Monument Hill on which stands a memorial to the men and women of Albury who died in the two World Wars. The provision of a landmark structure at the eastern end of this axis provides a suitable counterpart to that of Monument Hill.

The corridor to be spanned by the pedestrian bridge comprises the twin carriageway Hume Freeway, with three traffic lanes in each direction at this location, and the Great Southern Rail Line comprising eight tracks, six of which are in use.

Early concepts were of two arches supporting the walkway - one arch spanning the Freeway and the other spanning the railway tracks. The Design-Construct-Maintain (DCM) tender required the arch concept to be priced as the conforming bid, but Abigroup included a cable stayed alternative. The contract was awarded to Abigroup and the RTA during the post-tender discussions accepted the cable stayed arrangement as the bridge concept.

3. GENERAL DESCRIPTION OF BRIDGE

The pedestrian bridge spans across the Great Southern Railway Lines and the new Hume Freeway to link Dean Street with East Albury and the East Albury Cycleway. The bridge has two 67 metre spans with a central pier between the Rail and Freeway corridors. This is illustrated in Figure 1.

The bar stays are anchored at the top of the pier. Twelve pairs of stays support the superstructure and a further two pairs act as backstays. The stays are arranged in a semi-fan shaped configuration and were stressed at the lower ends.
The superstructure comprises a nominal 980mm overall depth twin cell steel box girder with a compositely acting reinforced concrete deck. The deck has a width of 4.05m between kerbs which are each 175mm wide at the top and 200mm wide at the underside. The clear width provided for pedestrians is 4.0m. A slip-resistant surfacing which also acts as a waterproofing seal was applied to the deck surface. Steel stubs passing through and welded to the box section were fabricated with anchorages for the lower ends of the stays.

Minimum vertical clearances of 7.36m and 5.5m were required over the Great Southern Rail lines and over the Hume Freeway, respectively. However, the bridge was detailed symmetrically in elevation to avoid detracting from its appearance, and the actual clearance over the Freeway exceeds the minimum requirement. The superstructure was also detailed on a vertical curve, again for aesthetic reasons.

The deck surface varies in thickness laterally to provide a two-way crossfall of 1% in each direction. Scuppers drain into 150mm diameter UPVC longitudinal drainage pipes concealed behind the parapets. These pipes connect the runoff to the local drainage system at each end of the bridge.

The bridge is supported on laminated elastomeric bearings at each abutment. Longitudinal and lateral restraint is provided at the pier while allowing vertical displacement under live load. Steel plate expansion joints are provided at each abutment.

The tower comprises two vertical columns, one on each side of the superstructure, with horizontal braces. The tower itself was designed in steel and is supported on and bolted to a reinforced concrete column stub which is designed for train impact loading. The column stub forms part of a pile cap which is in turn supported on 900mm diameter bored piles bearing on dense sand. The tower has an overall height of approximately 41.7 metres above the existing surface.
The eastern abutment comprises a reinforced concrete headstock supported on bored piles. The abutment is located on top of a reinforced soil embankment and is part of a podium area. A ramp continues down from the bridge to a pedestrian/cycleway through the East Albury Park. The eastern backstays are anchored to the abutment. There is no net uplift on the abutment piles, but the horizontal component of the backstay force is resisted by additional reinforced soil straps attached to the rear of the abutment headstock. Access pits with steel cover plates have been provided on either side of the abutment to allow for future jacking or replacement of the backstays. These pits also act as drainage pits for the longitudinal deck drainage from the eastern end of the bridge. Transparent screens are provided around the podium and terminate adjacent to the safety screens at the end of the bridge. These screens act as noise barriers and connect to the noise barriers between the Park and the Freeway. They also act as safety screens and minimise the risk of objects being thrown from the podium onto vehicles below.

The western abutment is similar and incorporates access pits for the backstay anchors which also act as drainage pits for the longitudinal deck drainage from the western end of the bridge. A ramp and stairs are provided on the reinforced soil embankment for pedestrian and cycle access to Dean Street.

Wire mesh safety screens with steel posts are attached to the sides of the parapets to prevent objects being dropped and to minimise the risk of objects being thrown from the bridge onto trains and vehicles below. Overhead lighting for pedestrians is fixed at the tops of the safety screen posts.

4. DESIGN LOADINGS

The bridge was designed to the requirements of AS 5100\(^{(1)}\) and is subject to the loadings in Section 2. The more significant loads were as follows:

- Self weight of steel and concrete
- Pedestrian loading of 5kPa SLS with reduction for loaded area in accordance with AS 5100.2
- Maintenance vehicle loading of 20kN SLS in accordance with AS 5100.2
- Wind loading with a design ULS wind speed of 48m/sec
- Earthquake loading with an acceleration co-efficient of 0.08 m/sec\(^2\), a site factor of 1.25 and a Type II bridge classification
- Rail collision loads on the base of the pier of 3000kN parallel to and 1500kN normal to the rail lines, since the pier is less than 10m from the nearest track centerline, acting 2m above rail level
- Rail collision load on the superstructure above the tracks of 245kN applied in any direction
- Minimum lateral restraint provisions of Clause 9 of AS 5100.2 at the abutment and pier supports. The 500kN ULS horizontal load exceeded the design effects of wind, earthquake or train impact
• Temporary loss or replacement of any single stay with a concurrent live load limitation of 2kPa SLS

5. BRIDGE CONCEPT AND DESIGN OBJECTIVES

A literature search provided the following parameters which were used to confirm the concept details:

• Minimum stay inclination \(25^0\)
• Recommended tower height (above deck) \((0.35 – 0.45) \times\) span
• First mode torsional frequency to be very much greater than first mode flexural frequency. A ratio of 2.5 is desirable

The last requirement is to ensure that the superstructure, with relatively long spans for a pedestrian bridge, was not susceptible to wind oscillation effects. The superstructure was detailed to comply with these parameters.

A concrete deck was detailed for optimum pedestrian comfort with respect to local vibration effects, to provide a better damping of global vibration and to minimise long term maintenance.

The superstructure was articulated to avoid being supported vertically at the pier in order to avoid a large negative moment at that location. It also avoided a reaction which would be sensitive to the superstructure and stay stiffnesses and temperature effects and which could vary significantly from the reaction derived from the design analysis.

It was appreciated that the maximum live load deflection of the superstructure would occur with live load on one span. In order to limit the SLS deflection under this loading to span/600, backstays were provided to anchor the top of the tower to the abutments. These not only helped to limit deflection, but they also limited the dynamic response of the superstructure under pedestrian movement, reduced the bending moment in the tower and prevented low reactions or uplift at the abutments.

After preliminary evaluation of the forces in the stays and selection of the stay diameters, effective moduli were evaluated for each stay allowing for the sag effect. Different values were required for the SLS and ULS load cases.

6. DETAILED DESIGN

6.1 Design Methodology

The structure was analysed using Microstran models for all relevant static loading combinations including railway collision loading on the pier and on the superstructure above the tracks.
A transient dynamic analysis was completed using STRAND7 to investigate compliance with the maximum dynamic amplitude limit specified in Section 12.4 of AS5100.2. Based on the published findings of Wheeler\(^4\), it is considered that the ordinates of Figure 12.4 in AS5100.2 should be divided by a factor of 10. The dynamic analysis of the Dean Street bridge showed compliance with this modified requirement. This is described in greater detail in Section 6.4.

STRAND7 was also utilised to evaluate local effects using finite element modelling for the stay anchorages at the top of the tower, the fabricated anchor beam connections to the box girder and the effect of train impact.

STRAND7 was also used to confirm the fabrication pre-camber profiles which considered the staged fabrication and erection sequence.

### 6.2 Option with 6 Pairs of Stays

The tender concept accepted by the RTA had 14 pairs of stays spaced at 10.3m. This concept had been based on preliminary design during the DCM tender period.

At the detailed design stage, it soon became evident that the 980mm depth of superstructure required to accommodate the fabricated anchor beams, which passed under the edge beams and the longitudinal drainage pipes and through the fabricated box girder, could span much further than 10.3m and a revised stay arrangement comprising 6 pairs of stays at 16.8m centres was adopted as shown in Figure 2. This better utilised the box girder section, generating a more conventional positive and negative bending moment envelope. It also utilised greater tension in the stays, resulting in less sag, and eliminated the flat angled stays nearest to the abutments which had very little tension and were virtually redundant.

![Figure 2 Elevation showing 6 pairs of Stays](image-url)
The design was completed on this basis and was submitted to the RTA for approval. However the 12 stay design was not accepted. The RTA requested that the design be revised back to the original concept with 14 pairs of stays as this was the offer by Abigroup and was the RTA’s preference aesthetically.

6.3 Final Design with 14 Pairs of Stays

The bridge was re-designed with 14 pairs of stays to satisfy the RTA’s aesthetic requirement.

The main issue was that the flat angled stays anchored to the deck nearest to the abutments had almost no self weight tension which resulted in significant sag. Increased jacking force improved the sag but reduced the reactions at the abutments. A compromise was adopted which resulted in a reasonable sag while maintaining a positive reaction at the abutment.

VSL MT600 bars varying in diameter from 30mm to 56mm were used for the stays.

Interestingly, as a result of the loading history and the large numbers of stays, the superstructure is almost entirely in positive bending under all load cases.

6.4 Dynamics

For pedestrian bridges with resonant frequencies for vertical vibration within the range 1.5 Hz and 3.5 Hz, AS 5100 requires the vibration of the superstructure to be investigated as a serviceability limit state. There is a further requirement where the fundamental frequency of horizontal vibration is less than 1.5 Hz, that the structural response to lateral excitation should be considered.

Although AS5100 describes the pedestrian load as a 700N load traversing the bridge at a given range of walking speeds, simulation of the traversing load at a fixed tempo is not easily handled by common structural analysis software. Hence the more popular approach is a simplified procedure recommended in BD37/01 involving excitation of the structure with a load applied at the midpoint. Many variations of the same method have been derived to account for limitations on articulation and bridge geometry as described by Wheeler, however few of these variations actually simulate the load traversing across the bridge.

The procedure adopted here follows recommendations by Wheeler and provides a more accurate simulation of the moving load described in AS5100 by means of the inbuilt transient dynamic capabilities of Strand7. This is considered to offer a more realistic representation of the moving pedestrian than the simplified methods.

A 3D beam model of the bridge was set up and a natural frequency analysis identified two vertical resonant frequencies of 1.80 Hz and 2.25 Hz within the range of concern as
well as a lateral resonant frequency of 1.61 Hz. Although the lateral frequency is above the recommended range of concern, the structural response to lateral excitation was nevertheless assessed out of interest.

A literature review was carried out to identify walking styles that were closest to the resonant frequencies of this bridge. In the aftermath of the unexpected pedestrian-excited vibration of the Millennium Bridge, Newland\(^3\) reviewed dynamic loading caused by pedestrians for both vertical and lateral vibration. He cites data from Bachmann and Ammann (1987) summarising the frequencies of applied load for a range of walking speeds, as follows:

<table>
<thead>
<tr>
<th>Walking speed</th>
<th>Forward speed (m/sec)</th>
<th>Stride length (m)</th>
<th>Vertical fundamental frequency (Hz)</th>
<th>Horizontal fundamental frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slow walk</td>
<td>1.1</td>
<td>0.60</td>
<td>1.82</td>
<td>0.91</td>
</tr>
<tr>
<td>Normal walk</td>
<td>1.5</td>
<td>0.75</td>
<td>2.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Fast walk</td>
<td>2.2</td>
<td>1.00</td>
<td>2.22</td>
<td>1.11</td>
</tr>
<tr>
<td>Sow run (jog)</td>
<td>3.3</td>
<td>1.30</td>
<td>2.50</td>
<td>1.25</td>
</tr>
</tbody>
</table>

The “normal walk” and “fast walk”, considered closest in pace to the vertical resonant frequencies, were applied as 700N vertical pulses at succeeding nodes positioned to coincide with stride lengths representative of these walking styles.

Vertical deflections were logged for time-steps/ intervals both in sync and out of sync with the pulses and at various points along the bridge to ensure that maximum values were captured. Based on recommendations by Wheeler, the model was run again with pulses synchronized to the resonant frequencies. It was observed that the maximum dynamic deflection was 0.127mm and applying the load at resonant frequencies did not produce any significant amplification.

The analyses were then repeated by applying 1% to 4% modal damping with the upper limit obtained from recommendations by Blanchard et al\(^2\). It was found that the dynamic deflections were not sensitive to damping in this particular situation. A possible explanation is that the changing position of the pulse is counterproductive to the movements it excites and the damping produced by this phenomenon is more pronounced than the applied modal damping. The dynamic deflection falls safely within the “corrected” limit set out in AS5100 and it was concluded that vertical vibration was not an issue.

A similar approach was applied to the analysis of lateral vibration. A load magnitude of 35N as recommended by Newland was adopted and applied at succeeding nodes in alternating lateral directions. The “slow jog” was investigated since every alternate footfall had a frequency of 1.25Hz, which was closest to the resonant frequency of 1.61Hz. The maximum lateral deflection recorded was 0.06mm. Based on the AS5100 dynamic deflection limit for vertical vibration, which was considered reasonable to adopt for lateral vibration, it was concluded that the structure was not sensitive to lateral vibration.
The effects of synchronized crowd excitation were not investigated as there are no current guidelines on how it can be quantitatively evaluated. Furthermore, results put forward by Arup in Newland show that the probability of synchronized crowd excitation is proportional to the bridge lateral amplitude which in the case of this bridge suggests that the probability of synchronized crowd excitation is low.

The observed response of the completed bridge being traversed by pedestrians appears to confirm that the bridge is not susceptible to vibration.

6.5 Aesthetics and Urban Design

Aesthetic objectives were established by the Urban Designer for this essential pedestrian and cycle link across the transportation corridor comprising the Hume Freeway and the Main Southern Rail Lines, and these objectives were addressed in the following ways:

- A visually distinctive bridge to be provided as a visual marker for this crossing - this was satisfied by adopting a slender cable stayed structure
- Main tower to relate to visual form of memorial on Monument Hill at the other end of Dean Street – the bridge tower was detailed vertical and painted white
- Bridge to provide a visual connection with East Albury – a landing and podium with transparent screens, a portal and Canary Island date palms was created on the eastern side to provide an elegant connection to the East Albury Park – wide stairs and a ramp provide the connection to Dean Street at the western end allowing level access to bridge
- The bridge is lit by glare controlled floodlights and also incorporates feature lighting to highlight the edges of the parapets and the tower
- The stays are connected neatly at the top of the tower using a forked clevis and clevis pin arrangement with a conical cover. Couplers in the stays were located a consistent dimension down from the tower connection
- The striking appearance and slenderness of the bridge in this rail yard setting has been emphasised by painting the tower and parapet faces white, and the fabricated steel box and the stays a charcoal colour. An open feeling is provided for pedestrians with the inclined safety screens provided only on the sides of the walkway and not as a full enclosure over the top

6.6 Durability and Protective Treatment

The bridge was designed and detailed for a 100 year design life. Concrete strengths and cover to reinforcement comply with AS 5100 and construction was specified to comply with normal RTA standards. Although it was confirmed that aggressive ground conditions were not present, provision was made for future cathodic protection in all of the reinforced concrete substructure elements.
The external surfaces of the steel tower, the fabricated box girder and the stays were painted with an inorganic zinc silicate primer, a mid epoxy second coat and a top coat of polyurethane.

As there was no possibility of future access within superstructure, and as the concrete deck slab could not be considered completely impervious to moisture ingress, a 6mm thick top plate was welded to the fabricated box section in order to fully seal the box. As a consequence, no protective treatment was required for the steelwork within the tower or the box girder as the interiors are completely sealed.

6.7 Erection Sequence

The design of the superstructure was based on a construction sequence which was developed in consultation with the Contractor and documented in the bridge drawings. After construction of the foundations, abutments and pier stub, the superstructure was erected as six segments on temporary falsework towers. This is shown in Figure 3. The towers were not at regular centres but were located to clear the rail tracks with a minimum horizontal clearance of 3.2m to the nearest live rail and to also clear the concrete paving works on the Freeway. The segments ranged in length from 18.2m to 29.8m, and the masses of the composite segments ranged up to 97 tonnes.

![Figure 3 Erection of Superstructure on Falsework Towers](image)

The steelwork for the superstructure segments and the tower was fabricated off-site. The steel superstructure segments were delivered to site and placed on temporary supports beside the bridge where the concrete decks were cast. The steel segments were continuously supported to ensure that the girder segments would act compositely under self weight. After erection and placement on the falsework towers, butt welding of the steel girder connections, local protective treatment of the welded connections and completion of the deck slab pours at the joints was carried out.
The steel towers were erected to straddle the girder segments with the stays already connected at the top. The lower cross brace and the restraint were then attached, and the stays were connected to the lower anchor beams and stressed to nominated loads calculated to just achieve lift-off at each of the towers.

7 CONSTRUCTION

The constructability of the bridge was progressively assessed by the Abigroup construction team during the course of the design. Specialist advice was received from VSL Prestressing regarding the erection of the stays which are typically attached to the top of the tower before it is erected. Open slots rather than holes were detailed at the ends of the fabricated anchor beams to avoid introducing curvature into the stays during installation. The erection sequence is described in Section 6.7.

Detailed construction safety plans and procedures were developed by the Contractor in view of the construction over and adjacent to an operating railway and over and adjacent to the continuing construction of the Freeway.
The megashore falsework tower arrangement during erection provided a 3.2m minimum horizontal clearance to the nearest live rails. The Contractor negotiated the sequence of erection and removal of the falsework towers with ARTC. A train speed restriction was implemented while the bridge segments were supported on the falsework towers and also the full time presence of a flagman.

8 CONCLUSION

This iconic pedestrian bridge is aesthetically pleasing and satisfies the strength, serviceability and durability requirements of the Client and AS 5100.

The success of this bridge project can be measured by its very favourable public acceptance. This was largely achieved by a community consultation process and excellent liaison between the designers, the construction contractor and the Roads and Traffic Authority. Following the completion of the Freeway, more than 20,000 people attended its official opening by the Prime Minister and a community open day on 4 March 2007.

9 REFERENCES

3) NEWLAND David E, “Pedestrian Excitation of Bridges – Recent Results”, Tenth International Congress on Sound and Vibration, Stockholm, Sweden, July 2003

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