Lane Cove Tunnel
Bridge Refurbishment for the Falcon Street Interchange, North Sydney

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SYNOPSIS

The Lane Cove Tunnel, Sydney, provides the final link in the Sydney Orbital road network. The Falcon Street Interchange is situated at the southern extremity of the project where Falcon Street crosses the Warringah Freeway between the suburbs of North Sydney and Neutral Bay. This project includes the introduction of three new ramps connecting to two existing bridges, bridge widenings to accommodate new pedestrian and cycle paths, reconfiguring of existing lanes at both the Falcon Street and the Warringah Freeway levels. This paper discusses the solutions implemented to overcome the complexities in the design and construction of the widenings and new ramps for this project.

INTRODUCTION

This project, undertaken as part of Sydney’s Lane Cove Tunnel project on behalf of the New South Wales Road and Traffic Authority, NSW (RTA), consists of a major reconfiguration of the Falcon Street/Warringah Freeway (WFWY) Interchange in North Sydney, Australia. The interchange is located approximately four kilometres north of the Sydney Harbour Bridge as shown in Photograph 1. Two existing bridges, built in the 1960’s, cross over 17 traffic lanes on the Warringah Freeway. The bridges lie on an east-west alignment along Falcon Street and are referred to as the East and West Bridges respectively. Existing ramps include the southbound WFWY entry ramp to the Sydney harbour bridge and tunnel, and the northbound WFWY exit ramp. For the West Bridge this project includes widenings to provide for pedestrian/cycle paths on south side and a pedestrian only path on the north side, and the introduction of two new suspended entry and exit ramps connecting to the north and south sides respectively. Similarly, for the East Bridge this project includes widenings to provide for pedestrian/cycle paths on south side and a pedestrian only path on the north side, and the introduction of a new exit ramp at grade connecting to the north side.

The existing bridges were designed for NAASRA H20-S16-44 traffic loads. The bridge widenings were therefore designed for the same loading and there was no requirement to retrofit the existing structure. The elevated sections of the new ramps were designed to the requirements of AS5100 with a provision for SM1600 loading. A requirement of the project was that all traffic lanes within the project area remain in operation during construction, except for limited night and midday lane closures. No long term temporary lane closures that were not part of the future final configuration were permitted. Figure 1 shows the arrangement of the reconfigured interchange. More detailed discussion of the project follows.
Photograph 1: Aerial views looking south (left), looking west (right)

Figure 1: Falcon Street Interchange showing new ramps
WEST BRIDGE

PROJECT REQUIREMENTS
The existing bridge comprised two separate parallel structures carrying 4 eastbound traffic lanes and 3 westbound traffic lanes with a footpath on the northern side. The main requirement was to introduce a new 3.5m wide footpath on the southern side. The resulting reconfiguration of the traffic lanes on the bridge required the two structures to be joined together. In addition new north and south facing ramps required additional structure to connect to the existing bridge. There was no requirement to strengthen the existing bridge elements to upgrade the load carrying capacity to current design standards.

EXISTING STRUCTURE
The existing structure comprises a non-composite reinforced concrete deck supported on continuous steel girders over three spans of approximately 25m, 31m and 21m respectively. The existing girders are 1500mm deep, spaced at 2500mm centres, and supported on steel rocker bearings. The existing non-composite deck slab is 200mm thick. The substructure comprises reinforced concrete abutments and piers founded in cuttings through Hawkesbury sandstone. On the northern side, copious existing services are suspended between girders and include a 375mm diameter watermain, power and communications conduits. These had to remain live for the duration of the construction phase. Vertical clearances between the freeway and the underside of the existing bridges were marginal and in some instances below the project requirement of 5.3m.

WIDENINGS AND RAMP CONNECTIONS
Figures 2, 3, and 4 show an elevation, typical cross section and the steelwork arrangement for the northern and southern widenings.

On the northern side, the widening required the demolition of the existing footpath in order to match the existing and adjacent proposed finished surface levels. This in turn required the removal of the top flange and the upper third of the web of the outermost steel girder which supported the footpath at a higher level compared to the adjacent main bridge girders. The remaining portion of the web of this girder was stiffened by welding a rolled steel angle to the web prior to the removal of the upper portion. One additional new longitudinal girder was introduced adjacent to the modified girder. The main purpose of the new girder is to support the connection to the new northbound entry ramp steelwork. On the southern side two new longitudinal plate girders were required to accommodate the new southern footpath as well as to provide support for the new northbound exit ramp structure. All new longitudinal girders incorporated shear studs to achieve composite action with the deck slab.

The existing piers and abutments were extended with sections to match existing. A combination of elastomeric fixed, sliding and pot bearings was used depending on the location in the structure. Ensuring compatibility in the deflections between the new and existing structure was an important consideration in the analysis and design.

The deck comprised 65mm thick precast concrete Transfloor panels reinforced with T8012 steel trusses at 450mm centres. A nominal 135mm in situ pour was placed giving an overall slab depth of 200mm minimum. Complex geometry, in particular on
the ramp connections, required careful shop detailing of these panels to ensure adequate seating on the girders and to avoid clashes with the shear studs located on the girders.

Figure 2: Elevation looking south

Figure 3: Typical cross section

Figure 4: Steel girder layout plans, northern widening (top), southern widening (bottom)
Prior to and during modification of the existing steelwork and demolition of existing deck and parapets, there was a constant flow of information between the construction and design teams to ensure that the design geometry fitted with the existing. This was partly because the existing structures could not be fully surveyed at the time the original design was carried out. As a result there was a significant number of readjustments of the design geometry and structural details to suit the existing geometry and structure.

Extreme care was exercised during fabrication to ensure that the new steelwork fitted the existing geometry. This was critical given that erection occurred during overnight lane closures with no opportunity to correct any significant geometric deficiencies that may occur. To this end the existing structure was meticulously surveyed and in addition the steelwork was fully assembled in the fabricators workshop for the full length of both widenings, and checked for geometric correctness. Some deficiencies were identified in the workshop and corrected prior to delivery to site. As a result only minor problems were encountered during the otherwise successful night erection of the steelwork with no adverse impacts on the construction programme. Girder erection was completed a span at a time with full lower carriageway closures.

On both widenings a nominal 600mm wide gap was left between the existing and new concrete pours to allow a later stitch pour. These are shown as the shaded portions shown in figure 4 above. On the northern side an additional provision was made for a wider stitch pour of 3.0m due to the relatively long central span and the need to maintain control over the relative deflections between the new and existing structures. This enabled the new structure to undergo its deflection prior to being connected to the existing structure. This also allowed for later access to install the cross bracing for this section. In addition a temporary prop was installed and left in place until such time as the stage 1 pour slab had achieved design strength and composite action. At this stage the prop was removed, the cross bracing installed and the stage 2 concrete pour was completed. The stitch pours were undertaken during weekend overnight lane closures when traffic was lightest. A minimum concrete strength of 20MPa was required prior to opening lanes and this was generally achieved in 12 hours using a specially designed concrete mix with a very high level of control on the target slump of 110mm. Slump tests were carried out on each truck load of concrete prior to being placed.

Photograph 2: North widening stitch pour
East Bridge

Project Requirements
The east bridge, like the west, required widenings on the north side to accommodate the end of the new southbound WFWY exit ramp, as well on the south side to provide for a new 3.5m wide footpath and cycleway. The northern widening is similar to that on the west and is not discussed further. The curved new southern footpath was designed as an independent structure. Figure 5 shows the layout of the new southern footpath.

Existing Structure
The existing structure is a single span composite steel girder bridge with a span of approximately 29m. The substructure comprises reinforced concrete abutments which are founded in the road cutting on Hawkesbury sandstone. The new southern widening was designed as an independent structure so that no modification to the superstructure was required. New abutments and extensions to the existing abutments were required to support the new footpath steel structure.

Southern Footpath Structure
The superstructure comprises twin steel plate girders laced together to form a torsionally stiff structure acting compositely with the slab over. The two girders are 1600mm deep plate girders with 350mm wide flanges with thicknesses varying from 25 to 50mm and a web thickness of 16mm. A single line of shear studs are incorporated on each girder to connect into the deck slab. The internal lacing members comprised rolled steel angles. The deck slab comprises precast Transfloor panels with T8012 steel trusses at 450mm centres and an in situ concrete pour to form a 200mm thick slab overall. Originally discrete pockets were allowed in the design of the precast panels for the protrusion of the shear studs but during construction this was changed to a continuous slot. This minimised the risk of clashes between the precast panel trusses and the shear studs. Notwithstanding careful detailing and coordination during the production of the structural steel and precast panel shop drawings was required to ensure that no clashes occurred. Figure 6 shows a typical section and a photograph of the steelwork in position.
The structural steel was fully assembled in the fabricators shop to ensure fit. Final erection occurred overnight and was done in two stages with the shorter span being placed first over abutments C, E and F. The splice location is located at the third point between abutments C and D.

The structure is supported on four abutments (refer figure 5). Midspan deflections between abutments C and D were kept within acceptable limits by the backspan between abutments C and E resulting in significant uplift forces at abutment E.

Abutment C comprises an extension to the existing abutment wall and includes the installation of a prestressing tendon as shown in figure 7 below. The existing abutment wall is 610mm wide and widens locally to 990mm in the zone where the prestressing duct is located. The extension comprises an in situ concrete pour which was then post tensioned to the existing wall by means of 19x15.2mm prestressing strands cored through the centre of the existing wall and anchored behind to the back of the abutment. The dead end anchor was located under an existing live bus lane. The anchor position was accessed by excavation which was carried out on weekends. A heavy steel plate was placed over the excavation to allow continued use of the bus lane. On completion the excavation was concrete filled and the pavement reinstated.

Abutments E (refer figure 7) and F were concrete pads constructed on the top of the existing rock cutting and anchored with fully encapsulated stressed ground anchors (abutment E) and passive galvanised grouted tie bars (abutment F).
Photograph 3: Precast formwork being placed, and southern footpath precast formwork in place

RAMPS

_Elevated Section_

The original proposal was for the northbound WFWY entry and exit ramps to be constructed out of reinforced earth. The proximity of the adjacent traffic lanes made access for the construction of RE walls very difficult. In addition it was felt that RE walls would result in a “boxed in” feeling which was not considered a desirable urban design outcome. Consequently it was decided to elevate the ramps on piers spaced at 15m centres and to construct the deck out of 535mm standard prestressed planks with a 170mm in situ deck slab. The minimum clearance from the soffit of the deck planks to the finished surface below was around 2.0m to 2.5m at which point abutments were introduced. Beyond the abutments the ramps continued at grade with vertical retaining walls. Figure 8 shows a typical elevated ramp elevation.

![Figure 8: Typical Ramp Elevation – Northbound Entry Ramp](image-url)
**Ramp Retaining Structures**

One of the most difficult issues with the ramps was construction of the retaining structures between opposing carriageways, with live traffic on either side and in a tight time frame. The preferred solution had to include retaining wall panels which were precast off site and placed on in situ reinforced concrete bases and/or levelling strips. A further benefit of precasting was that the required surface finish and ribbed profile required by the urban designers could be achieved to a high standard. To this end three types of retaining wall were developed to suit conditions at individual locations. Figure 9 shows typical cross sections of the three types of retaining wall. For all three wall types, the wall is laterally restrained at the top to the pavement slab. The containment barriers were all precast and are connected to the pavement slab. Barrier impact loads are resisted by the pavement slab and not by the respective retaining walls.

Retaining wall type 1 is a conventional cantilever retaining wall in principle but with a precast wall stem. A base slab was placed in the normal way and the precast wall stem was attached by means of a later stitch pour. Placement of the reinforcement protruding from the panel and the base slab had to be carefully coordinated to avoid clashes during placing of the panel. Temporary props were required until such time as the stitch pour concrete had achieved the required strength.

![Figure 9: Typical Retaining Wall Sections](image)

Retaining wall type 2 was developed to accommodate the vertical rock face below the level of northbound entry ramp. The precast wall section is up to 5m high and is supported at the base on a concrete strip footing socketed into sandstone rock. After placing and temporarily propping the wall panel, the narrow void between the back of the wall and the rock face was filled in stages with flowable cement stabilised backfill up to the level of the rock ledge. The base slab was anchored to the rock cutting and provided support at the mid height of the wall panel. On completion of the base slab the temporary props could be removed and backfilling completed.
Retaining wall type 3 is similar to type 2 except there is no conventional backfilling required above the level of the rock. Consequently a galvanised rock anchor ties the wall panel to the rock cutting prior to filling the narrow void between the rock face and the wall panel with flowable cement stabilised backfill.

CONSTRUCTION ISSUES
The most significant issue relating to the construction of this project was the requirement to carry out all construction activity adjacent to and/or over live traffic without any long term lane closures. Lanes could be relocated, however only within the tight constraints imposed by the existence of multiple traffic lanes, existing structures and the sandstone cuttings. To overcome these constraints the design and construct team had to work very closely together during design and in particular during construction. Alternate designs, which would not otherwise have been considered, had to be fully investigated with the daily interchange of detailed information between the site and design offices being a common occurrence. The designs adopted included dispensing with the initial proposal for the ramps to be constructed of reinforced earth, and replacing these with precast retaining wall panels anchored into rock and/or stitch pours to connect into base slabs. The ramps were elevated on piers and abutments with precast plank decks used to minimise on site works. All elevated parapets were precast and deck slabs were generally placed on permanent precast formwork.

Ongoing construction issues included the redesign of minor structures including sign gantry footings, street light pole footings with almost every footing being unique to suit the particular location in which it was located. The same issues occurred with other elements including drainage, street light and traffic signal post placement to name but a few. Traffic signal posts with significant cantilever overhang up to 16m had to be retrofitted to the bridge deck using heavy steel base plates and chemical anchors.

A further significant construction issue is that the detailed design was carried out remotely with the majority of the design team located in Brisbane, Queensland. This created the need for very careful coordination and interfacing. Most members of the remote design team were not fully familiar with the site and the issues that needed to be addressed in the design to ensure constructability. This was managed by regular site visits by key members of the design team during both the design phase and in particular during the construction phase.

CONCLUSION
The complexity of the design and the site conditions meant that many interfaces only became evident as construction proceeded. The design and construction teams were integral from beginning to end, working hand in hand resolving issues of design, construction and constructability, while also complying with the project procedural requirements and approval processes.