CityLink Tulla Freeway Widening Project  
Widening of Existing Moonee Ponds Creek Bridges  

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1 ABSTRACT  

As part of the CityLink Tulla Project, widening was undertaken to accommodate additional traffic lanes to a number of existing structures including four separate bridge crossings over the Moonee Ponds Creek. Load rating of existing structures was also undertaken and strengthening implemented where required to achieve the loading requirements. While all structures were similar single span bridges, each presented its own set of constraints and challenges and hence unique solutions were developed for each location.

This paper presents the key aspects of the bridge widening, load rating and strengthening undertaken, along with some of the challenges and outcomes associated with the works.

2 INTRODUCTION  

The CityLink Tulla Freeway Project incorporates additional traffic lanes and other measures to improve traffic flow along approximately 19 kilometres of CityLink and West Gate Freeways between Power Street and Bulla Road, Melbourne. The project ultimately increases capacity of the roadway by at least 30 percent between the Melbourne CBD and Melbourne Airport, reducing congestion and travel times and increasing safety for motorists with new lane-use management systems. Aurecon partnered together with GHD to deliver the detailed design of the freeway widening for CPB Contractors and continued to provide technical assistance during the construction phase.

Modification of a number of existing structures was undertaken as part of the project to accommodate additional traffic lanes, including widening of four separate bridge crossings over the Moonee Ponds Creek. These works included widening of superstructures and substructures in various configurations, along with construction of ancillary items including retaining walls, upgraded bridge parapets, off-structure barriers, noise walls and approach structures. Load rating analyses were also undertaken for the existing structures and strengthening works constructed to achieve the load requirements.

This paper presents the key aspects of the bridge widenings at the four separate locations, load rating and strengthening undertaken, along with some of the challenges and outcomes associated with the works.
3 EXISTING BRIDGES DETAILS

Existing Moonee Ponds Creek Bridges 1, 2, 4 and 6 carry CityLink Freeway inbound and outbound carriageways over the creek via a single, simply-supported span and are located within the at-grade portion of project. The original structures were constructed as part of the Tullamarine Freeway in the 1960’s and were subsequently widened on the eastern (inbound) side and strengthened as part of the Melbourne City Link - Western Link project in the 1990’s. Key details of the existing bridge structures are presented below.

Moonee Ponds Creek Bridge 1 (MPC1)

The original 1960’s bridge comprises two individual superstructures each with an approximate span of 23.2m and skew of 43. The superstructures comprise fourteen 1,450mm wide × 1,220mm deep post-tensioned segmental box girders acting compositely with a 100mm thick cast in-situ concrete deck slab, with girders supported on a pair of laminated rubber bearings at each end. Transverse diaphragms are included at abutments and quarter points. The superstructures are supported by continuous parallel blade wall abutments at each end founded on spread footings which are nominally embedded into the underlying rock.

The 1990’s widening comprised 9 additional 1,500mm deep pre-tensioned Super-T girders on the eastern side of the original structure and an additional 2 Super-T girders to fill the gap between the two original superstructures, all seated on elastomeric bearings as shown in Figure 1. Load was jacked from the original box girders into the new intermediate Super-T girders to relieve load from the original girders and a 170mm thick cast in-situ concrete deck slab tied the original and widened superstructures together. The 2 intermediate Super-T girders are supported on the original abutment and the 7 girders on the eastern side supported by an extended blade abutment with spread footings. Original barriers on the outbound side were replaced with new barriers as part of the work.

Strengthening for HLP loading comprised installation of vertical suspension bolts straddling each transverse diaphragm at the first joint between original girders either side of the new girders.

Figure 1 Moonee Ponds Creek Bridge 1 Existing Structure
Source: Melbourne City Link Project Western Link Tullamarine Freeway drawings
Moonee Ponds Creek Bridge 2 (MPC2)

The original 1960’s bridge has an approximate span of 25.7m and skew of 18 degrees. The superstructure comprises 27 pre-tensioned 1,143mm deep I-girders acting compositely with a 150mm thick cast in-situ concrete deck supported on laminated rubber bearings. Transverse diaphragms are included at two intermediate points. The superstructure is supported by two non-parallel blade wall abutments that are founded on pile caps and raked 500mm diameter driven piles with central rock tension anchors for rear piles.

The 1990’s widening comprised 9 additional 1,200mm deep pre-tensioned Super-T girders with composite cast in-situ concrete deck slab tied into the existing deck. Girders were supported on elastomeric bearings installed on abutments. Original blade wall abutments were extended with new blade abutment walls founded on pile caps with precast concrete driven piles. Original barriers on the outbound side were replaced with new barriers as part of the work.

Strengthening for HLP loading comprised installation of shear studs through the deck slab into the existing girder webs below and construction of a new transverse diaphragm at mid-span.

Moonee Ponds Creek Bridge 4 (MPC4)

The original 1960’s bridge has an approximate span of approximately 22.5m and skew up 22 degrees. The superstructure comprises 15 pre-tensioned 1,143mm deep I-girders with deck and bearings as per MPC2 and substructure as per MPC1.

The 1990’s widening comprised 9 additional 1,200mm deep pre-tensioned Super-T girders with bearings, deck, abutments and upgraded barriers as per MPC2.

Moonee Ponds Creek Bridge 6 (MPC6)

The original 1960’s structure has an approximate span of 24.8m and skew of 30 degrees. The superstructure comprises 17 post-tensioned 1,143mm deep I-girders
with deck slab and bearings as per MPC2. The superstructure is supported by two parallel blade wall abutments founded on pile caps with three rows of 350 mm square raked driven piles with tension rock anchors on rear piles.

The 1990’s widening comprises 9 pre-tensioned Super-T girders ranging from 1,200mm to 1,500mm deep with bearings, deck slab and substructure as per MPC2.

4  BRIDGE WIDENING

Existing Moonee Ponds Creek Bridges were widened on the western (outbound) side to cater for additional traffic lanes as part of the CityLink Tulla Project.

4.1  Bridge Widening Details

While the widening width varied for each structure, all bridges were widened with additional Super-T girders seated on elastomeric bearings at each end and in-situ concrete deck slabs connecting to the existing decks. Existing substructures were modified in various configurations and ancillary elements constructed including approach slabs, wing walls, retaining walls, medium performance bridge barriers, off-structure barriers and noise walls. Expansion joints were replaced for the full width of the bridges including the new and existing decks. Unique structural details for each of the bridge widenings are presented below.

Moonee Ponds Creek Bridge 1

The superstructure was widened a varying width from approximately 1.6m to 2.9m with a single splayed 1,550mm deep Super-T girder supported on elastomeric bearings and 230mm thick in-situ deck slab. The existing deck slab was broken back above the existing edge box girder to lap new transverse deck reinforcement for connection of the widened deck.

The existing abutment wall was locally demolished and a widened reinforced concrete abutment crosshead constructed in front of an offset pile cap and single 1,200mm diameter bored pile installed within the fill behind the existing abutment wall, minimising the extent of demolition required to the existing abutment. The new abutment crosshead was connected to the existing structure with drilled dowel bars bonded with epoxy. Soil straps were attached to the widened abutment pile cap to resist horizontal forces and reduce the size of bored piles.

Approach slabs were dowelled to existing approach slabs and used to support the first off-structure barrier unit each side of the bridge avoiding the need for additional wingwall structures.
Moonee Ponds Creek Bridge 2

The superstructure was widened approximately 3.5m using two 1,350mm deep precast Super-T girders supported on elastomeric bearings with a 180mm thick in-situ deck slab. New girders were lifted above the level of the existing girders and cross fall reversed to minimise the thickness of asphalt on the new structure resulting from the previous carriageway cross-fall reversal. The existing deck slab was saw-cut and kerb upstand broken back to allow positive connection via extension of new transverse reinforcement across the top of the existing deck and vertically embedded ligatures installed with epoxy.

Exiting reinforced concrete blade wall abutments and pile caps were extended, supported by two 900mm diameter bored piles at each abutment. The foundation at one abutment was revised to three 750mm diameter bored piles during the construction phase to avoid existing piles located on site after excavation and demolition. Existing blade abutment walls and pile caps were saw-cut vertically and demolished down to pile level to construct the new works. Existing driven piles were partially demolished and reinforcement cut back to achieve physical isolation to the new pile caps and piles. New pile caps and abutment blade walls were stitched to the existing structure with embedded bars installed with epoxy. Soil straps were installed as per MPC1.

Existing wing walls were demolished down to pile cap level and new 6m tall L-shaped retaining walls constructed, founded on top of existing pile caps. Use of L-shaped retaining walls eliminated the need for piles which would have been difficult to install around existing driven piles, minimised the extent of demolition required and provided support for off-structure barriers, further minimising pile requirements. A soil nail wall required to facilitate abutment demolition was detailed as a permanent structure to minimise lateral load applied to the retaining wall, as outlined in Section 4.2.
Existing approach slabs were located under deep asphalt so the widened approach slabs were detailed to be independent to avoid the need for a doweled connection between the two. A geomembrane was installed across the interface between the existing and new approach slabs to control reflective cracking in the pavement above.

**Figure 4 Moonee Ponds Creek Bridge 2 Widened Structure**

**Moonee Ponds Creek Bridge 4**

The superstructure deck was only required to be widened by approximately 400mm, however the existing edge girder would have been overloaded from the additional traffic load applied approximately 1.2m beyond the existing inwardly-located barrier and the weight of new barriers and noise walls. To eliminate the need for abutment widening and associated modification of wing walls, the existing edge I-girder was removed and replaced with the new 900mm deep Super-T girder to suit the available depth seated on elastomeric bearings and with 200mm thick in-situ deck slab. The existing deck slab was broken back to retain and lap existing deck reinforcement into the new deck.

The top of the existing abutment was locally demolished and reconstructed with a new fender wall to provide a wider bearing ledge for installation of elastomeric bearings. The ledge was set down from the existing ledge to allow for installation of the Super T girder, elastomeric bearing and pedestal within the constrained depth. The new abutment elements were stitched to the existing structure with drilled dowel bars embedded with epoxy. The remainder of the concrete abutment, wing walls and spread footings was retained, with a concrete thickening cast over the toe of the spread footing under path level to enhance shear capacity and reduce flexure in the abutment wall stem above. A reinforced concrete tie-back was constructed between the widened abutment bearing ledge and the adjacent off-structure barrier pile cap to resist transient lateral loads applied to abutment. The tie-back eliminated the need for a concrete thickening on the face of the abutment wall which would have impacted flow capacity within the creek below.

Approach slabs supporting off-structure barriers were constructed as per MPC1.
Moonee Ponds Creek Bridge 6

The superstructure deck was widened a varying amount between 3.1m and 4.6m comprising two 1,200mm deep precast Super-T girders seated on elastomeric bearings and a 180mm thick cast in-situ deck slab. The existing concrete deck was saw-cut, exposed reinforcement epoxy coated and starter bars drilled and embedded using epoxy due to the limited existing reinforcement for a positive connection.

Existing abutment walls were extended as per MPC2, supported on precast concrete driven piles and pile caps. New driven piles to be installed within the influence zone of existing piles were sleeved and predrilled prior to installation to eliminate lateral load transfer during pile driving.

Approach slabs supporting off-structure barriers were constructed as per MPC1.

4.2 Key Structural Design Considerations

Some of the key considerations associated with the design of the widening structures were as follows:

Modelling of deck connection

Connection of the widened deck slab to the existing slab was made by positive reinforcement connection where possible (MPC1, MPC 2 and MPC 4), or saw-cutting the existing slab and drilling and embedding of reinforcement starter bars using epoxy (MPC6). Use of epoxy was accepted by the Authority at this structure due to embedded bars being predominantly in shear rather than sustained tension and an
an additional reduction factor of 0.5 was applied to the capacity in accordance with VicRoads Bridge Technical Note BTN2012/02.

For MPC1 and MPC2, the interface between the existing and new deck slabs was modelled as a moment connection due to the depth available for extension of two layers of reinforcement. For MPC4, the low yield strength and volume of mesh reinforcement in the existing deck resulted in inadequate transverse flexural capacity at the connection to the new deck, hence the connection was modelled as both a moment resisting and pinned connection and superstructure elements designed for the worst case effects. The pinned connection resulted in reduced load sharing between the new and existing structure and was critical for the design.

For MPC6, the deck demolition line was close to the edge of the existing deck so lap lengths could not be achieved and existing mesh reinforcement was extremely light so welding was considered impractical. The existing deck was therefore saw-cut and a central layer of galvanised reinforcement bars embedded into the existing structure with epoxy. This bridge was also modelled as both a moment resisting and pinned connection due to the central row of dowels providing some moment capacity.

**Staging of deck construction**

Construction of the widened deck slab was detailed in two separate stages at all structures to minimise load transfer to the existing deck. The first stage pour was made over the new beams with an 800mm wide gap left to the existing deck, allowing a second stage stitch joint to be constructed later. The diagram below shows a typical staged stitch pour arrangement.

![Diagram of deck construction staging](image)

**Figure 6 Deck construction staging at Moonee Ponds Creek Bridge 6**

The second stage stitch pour provided the longitudinal and lateral connection between the existing and widened decks, transferring the effects of live load, longitudinal force
resulting from restraint to differential shrinkage and creep due to different ages of the concrete, effects of differential settlement and barrier impact load.

Time delays between the two pour stages minimised load transfer to the existing deck, allowing the early stages of creep and shrinkage within the widened structure to occur before the connection was made. A four-week minimum time delay was nominated between the stitch and first stage deck pour using VicRoads VR400/40 concrete mix with a 56-day shrinkage strain of 750 micro-strain. An alternative was provided for a reduced two-week delay using a low shrinkage 450 micro-strain mix.

**Traffic restrictions during deck construction**

During construction of the first stage deck pour the new deck was completely isolated from the existing, hence no traffic restrictions were imposed on the existing structure. The existing freeway carriageway remained open to traffic with vehicles running in the lane adjacent to the works zone behind temporary traffic barriers placed at a minimum offset from the demolition line.

During the second stage, lapped reinforcement between the existing and new widened deck slabs was tied together and the concrete stitch poured. Relative deflection between the existing and widening structure therefore needed to be limited during the second stage to avoid issues due relative movement of the decks or vibration including dislodgement or de-bonding of reinforcement. A relative deflection limit of 2mm was imposed between the existing and widened deck slabs, considered not to impact bond development of deck starter bars during curing, with any possible de-bonding effects or voids being less than the height of reinforcement bar deformations.

Traffic lane configurations on the existing structure were analysed to determine the required offset of lanes from the concrete stitch to ensure the maximum deflection limit was not exceeded. The existing superstructure was modelled without the widening and design traffic lanes progressively moved away from the stitch until live load deflection at the deck edge was acceptable. Deflections were assessed for serviceability limit state using a dynamic load allowance of 1.1, representative of the 40km/hr speed limit.

The minimum strength required for the stitch pour was calculated to determine the minimum strength required prior to removal of additional traffic lane restrictions. Crack widths were also checked to ensure they were less than the 0.2mm limit for crack repair outlined in the Project Specification.

Traffic restrictions offsets, and minimum concrete strength were identified on design drawings. At MPC2 for example, the temporary restriction was for no traffic lanes within 4.6m of the demolition line until a strength of 30Mpa had been achieved in the stitch pour, after which traffic could be reinstated back to the face of original temporary construction zone barriers and original speed limits reinstated.
Existing abutment demolition and temporary ground retention

Existing abutment blade walls were required to be saw-cut vertically and demolished over the full height of more than 6m to allow construction of the new abutment walls. Existing approach slabs were saw-cut and demolished and sheet piles installed or temporary soil nail walls constructed to retain the existing fill and carriageway beyond within the constrained area. The temporary ground retention systems accommodated demolition of existing abutments, wing walls and pile caps and construction of new abutments, wing walls and retaining walls. Stabilised sand backfill was then placed between the temporary ground retention and the new abutments and retaining walls.

At MPC2, the soil nail wall was designed as a permanent structure to retain lateral soil and surcharge loads applied behind the wall. Isolation material was installed to the front face of the soil nail wall and a tall L-shaped retaining wall constructed in front of the wall, with cement stabilised sand being installed in stages in the gap between the two structures. This arrangement minimised lateral load transfer to the retaining wall, ensuring stability of the narrow yet tall wall and eliminating the need for extensive piling which would have been difficult to install around existing piles.

Figure 7 L-Shaped retaining wall and permanent soil nail wall at MPC2

Cement stabilised sand behind abutments

Cement stabilised sand was used to backfill behind blade wall abutments, wing walls and retaining walls at all bridges. The stabilised sand was placed in stages comprising
limited layer depths, with each layer achieving a minimum strength prior to installation of subsequent layers, minimising lateral load on the new structural elements. The reduced load allowed for optimised structural design, reduced foundation requirements and also minimising load transfer to the adjacent existing substructure. 5% cement stabilised sand was used generally with 7% used in the upper layers to support soil straps in accordance with requirements of the strap supplier. Stabilised sand was also preferred by the construction contractor due to difficulties in installation and compaction of backfill in the deep and localised excavation with constrained access.

5  BRIDGE LOAD RATING & STRENGTHENING

Load rating of the existing structures was undertaken and load rating factors calculated in accordance with AS5100.7-2004: Bridge Design, Part 7: Rating of Existing Bridges. Ratings considered T44/L44 and 62.5t B-Double vehicles applied to the original structure and either the same loading or SM1600 applied to the widened structure depending on the widened width, as required by the Project Specification. Load ratings used structural models of the new widening together with the existing structures and considered the staged construction sequence. Rating factors were calculated for each major structural element except for the piles were serviceability design loads were compared to the working load limits defined on original as-built drawings.

The assessments were undertaken using as-built drawings and existing elements were assumed to be in sound condition and free from defects, with defects identified during inspection being repaired as part of the works.
5.1 Structural Deficiencies and Strengthening Details

A number of existing box girders at MPC1 returned unsatisfactory load rating factors for flexure. Flexural strengthening of box girders was undertaken using carbon fibre reinforced polymer composites (CFRP) epoxy-bonded to girder soffits as shown in Figure 9. CPB worked with the Supplier to develop installation procedures under live traffic which met the requirements as demonstrate by testing.

![Figure 9 CFRP Strengthening at Moonee Ponds Creek Bridge 1](image)

Interface shear capacity between the original I-girder and cast in-situ deck was found to be inadequate at MPC 2, MPC 4 and MPC6 for horizontal shear transfer at the ends of all I-girders not previously strengthened. Interface shear capacity was considered necessary to provide full composite action between I-girders and cast in-situ deck slab, as was required for satisfactory load ratings of exiting I-girders. Strengthening to enhance interface shear capacity was therefore required and comprised embedded reinforcement drilled and epoxied from the deck into the girder below as shown in Figure 10.
Figure 10 Interface shear strengthening at Moonee Ponds Creek Bridges 2, 4 and 6

Abutments at MPC4 were found to be inadequate for flexural capacity at the stem of the blade walls and for shear capacity in the spread footing toe. A reinforced concrete thickening was constructed over the top of spread footing toe which was connected to the existing footing and abutment wall using embedded reinforcement installed with epoxy.

5.2 Key Strengthening Design Considerations

Lateral loading on abutments

Existing blade wall abutments were tall, slender and lightly reinforced resulting in high loading and low strength in the wall stem and spread footing toe. Nearby boreholes were used by the geotechnical designer Douglas Partners to determine pressure coefficients and densities for evaluation of lateral loads for the analysis. Initial load rating using those loads resulted in unsatisfactory load ratings for abutments at MPC 2, 4 and 6. Furthermore, strengthening of substructure elements comprised thickenings of the top of spread footings and also to the front face of abutment blade walls which adversely impacted flow capacity within the creek below.

To reduce the scope of strengthening, a two-dimensional finite element soil-structure interaction model of the existing abutment was developed in modelling program RS2 by the geotechnical designer Douglas Partners as shown in Figure 11. The model which considered the time-history of the original construction was used to determine more refined lateral soil pressures applied to the abutment walls and vertical soil reactions under the toe of spread footings. The model considered elastic and non-elastic soil behaviour and reflected the construction staging of the original abutments. The assessment resulted in reduced lateral soil pressures which were approximately equivalent to a soil pressure coefficient of 0.4.
The reduced pressures were applied into the structural model and, along with increased capacities due to material testing, resulted in significantly enhanced load rating factors for abutments, limiting the scope of abutment strengthening to MPC4 only. Furthermore, as design actions in the abutment were reduced, strengthening was simplified to comprise thickening of the abutment spread footing only below path level, avoiding the need for thickening of the abutment blade wall and eliminating impact on the Moonee Ponds Creek.

**Modified compression theory**

In an attempt to further reduce the extent of abutment strengthening required, Modified Compression Theory was used to determine shear capacity in abutment elements in accordance with the DRAFT AS5100.5, available at the time of design. While this design standard was in draft format at that stage of design, the revised theory was in line with the approach outlined in AASHTO LRFD Bridge Design Specification and Canadian Highway Bridge Design Code and was considered a more refined assessment of shear capacity for reinforced concrete elements.

Shear capacity calculated for abutment elements using the revised theory was only nominally enhanced compared with that determined using the beam shear method from AS5100.5-2004: Bridge Design, Part 5: Concrete, and hence did not provide significant benefit for these bridges and was not formally adopted. Instead, strengthening requirements were reduced via increased concrete compressive strength observed from material testing and reduced lateral loads determined from the soil-structure interaction modelling, as designed below.

**Material properties**

Concrete strength for existing bridge elements was initially determined from original as-built drawings which indicated strength of 28Mpa for in-situ elements. The low strength resulted in low shear capacity which contributed to unsatisfactory load ratings of blade wall abutment stems and spread footings.
Due to the age of existing abutment elements and anticipated higher strength than reflected on as-built drawings, concrete cores were taken and compressive strength tests undertaken in accordance with AS1012-2014: Methods of Testing Concrete and AS3600-2009: Concrete Structures, Appendix B Testing of Members and Structures. The tests resulted in significantly increased concrete compressive strengths of 38Mpa to 46Mpa, enhancing capacity and contributing to the reduced strengthening requirements.

**Anchorage of beam tensile reinforcement**

Anchorage of tensile reinforcement developed beyond the face of bearing support was assessed in existing I-girders to ensure the required tensile capacity equal to 1.5V* had been achieved, as per Clause 8.1.8.3 of AS5100.5-2004. This requirement was determined to be unsatisfactory for the existing edge I-girder at MPC2. Strengthening of the girder for end anchorage was difficult to install and, given the gap between the beam and the fender wall was minimal, modification of the abutment fender wall may have been required to allow the strengthening to be constructed.

Anchorage of tensile reinforcement was re-evaluated using Clause 8.2.9.3 of the DRAFT AS5100.5 available at the time of design, later confirmed to be consistent with Clause 8.3.1.3 of the revised AS5100.5-2017: Concrete Structures. This revised approach considers the development of tensile reinforcement beyond an inclined line extending from the face of bearing support at the angle of the concrete compression strut ($\theta_v$), along with contribution from transverse reinforcement and vertical component of pre-stress force. This revision resulted in an increase in the effective development of inclined pre-tensioned strands and satisfactory anchorage requirements for tensile reinforcement and hence strengthening was not required.

6 Conclusion

Moonee Ponds Creek Bridges 1, 2, 4 and 6 were widened as part of the CityLink Tulla Freeway Project to accommodate additional traffic lanes and the existing structures were load rated and strengthened where required to achieve the loading requirements. While all structures were similar single span bridges crossing the Moonee Ponds Creek, each structure presented its own set of constraints and challenges and, as such, unique solutions were developed for each structure.

Bridge superstructures were widened between 0.4m and 4.6m with Super-T girders ranging in depth from 900mm to 1,550mm. Connection of the widened decks to existing structures was made with positive reinforcement connection where possible, or alternatively starter bars drilled and embedded with epoxy. Deck construction was undertaken in two separate stages to minimise impact on the existing structure and traffic staging and restrictions determined to allow continuous traffic flow on the bridges throughout construction while ensuring structural integrity of the deck connection. Modifications to existing blade wall abutments varied, ranging from extended blade walls supported on bored or driven piles, construction of bearing
ledges with offset pile supports, or localised bearing ledge modifications together with tie-beams and thickening of existing spread footings.

Load rating of existing superstructure elements identified deficiencies for flexural capacity in box girders at MPC1 and interface shear capacity between original I-girders and cast in-situ deck slab at MPC2, MPC4 and MPC6. These deficiencies were rectified using CFRP strengthening of box girder soffits and reinforcement starter bars embedded from the deck slab into the I-girder below. Strengthening to achieve the requirements for tensile reinforcement anchorage in original I-girders was avoided using the revised method contained within the DRAFT AS5100.5 and subsequently revised AS5100-2017.

Load rating of existing substructure elements identified deficiencies in the existing abutment blade wall stem and spread footing toe at MPC4 and strengthening comprised a concrete thickening of the spread footing similar to the widening design. More extensive strengthening was avoided by development of a finite element soil-structure interaction model to determine more refined lateral soil pressures, along with concrete sampling and testing to confirm increased compressive strengths than those shown on original as-built drawings.

7 AUTHOR BIOGRAPHY

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Chris has been part of Aurecon’s Bridges team for 10 years and has worked in both leadership and technical design roles across a range of infrastructure projects including CityLink-Tulla Freeway, Warragul Rail Precinct, Peninsula Link Freeway and Pacific and Oxley Highways. Chris recently returned from two years in Aurecon’s Bangkok office where he led teams to deliver project in various countries.