Bridge (East) Street Rail Underbridge Replacement for the Albury Wodonga Hume Freeway Project

John Steele BE (Hons 1) M Eng SC, Associate SKM Sydney
Gillian Sisk BEng PhD MIEI CEng, SKM, Sydney
Andrew Deck BE (Hons 1), MIEAust, CPEng, JP - Project Engineer Abigroup NSW

Synopsis
The Bridge Street Rail underbridge on the Great Southern Rail line in South Albury had to be replaced as part of the upgrade of local roads to link in with the Albury Wodonga Hume Freeway Project. The railway and road had to remain fully operational throughout the construction with only two short track possessions available to install the major bridge components. This possession regime led to a number of innovative construction solutions to install the bridge.

Introduction
Bridge Street was a two lane road to the south of Albury Railway station that linked East Albury with South Albury. It got its name from the 10m span steel railway underbridge that carried the Great Southern Rail line over the street.

In 2005 Abigroup were awarded the contract to design and construct the NSW section of the Albury Wodonga Hume Freeway from the Roads and Traffic Authority of NSW. Sinclair Knight Merz (SKM) were Abigroup’s design consultant for the project. As part of the project, Bridge Street became the major entry point to Albury City from the south and was widened to 4 lanes plus footpaths giving a bridge span of 25.2 metres. The vertical clearance to the bridge was also increased from 4.3 metres to 5.3 metres.

The bridge was originally to be installed in a single 4 day track possession at Easter 2006 but due to delays in other work on the line this possession was cancelled by ARTC. Instead, the bridge had to be constructed using a series of minor track occupancies between trains, a two day track possession at Christmas 2005 and another two day track possession during Easter 2006.

Figure 1: Bridge Street Prior to the Upgrade
This paper describes the construction solutions that were developed to install the bridge in the revised timeframes.

**Background**

The existing rail bridge was a 10m span transom top steel girder bridge with brick abutments which supported three railway lines: a disused standard gauge track, the standard gauge line from Sydney to Melbourne and a broad gauge line.

The concept design in the tender was to construct a steel through girder bridge on the alignment of the disused track and shift the standard gauge track onto this alignment then take the broad gauge line out of service and bus passengers from Wodonga station whilst constructing a second steel through girder bridge on the existing broad gauge track alignment. This concept was rejected during detailed design due to the 600 metres of new track and signalling required to bring the standard gauge track back onto its existing alignment at the required design speed. The escalating steel prices in 2005 also provided an incentive to look for alternate options. The design team was instructed to design a twin track concrete bridge on the existing operational track alignments.

The main 25m span over Bridge Street had to be a through girder because of the vertical clearance constraints to the street. Options for lowering of the road to provide the additional clearance were not possible due to extensive public utilities in the road corridor and drainage / flooding issues associate with creating a trapped low point at this location.

The early design development was for a three span bridge with short precast pretensioned plank back spans to spill through abutments. This was ruled out when the 4 day track possession was cancelled and replaced by two shorter track possessions. In order to construct the bridge using two short track possessions it became clear that the bridge had to be changed to a single span bridge and the completed superstructure had to be slid into position during the second possession.

The following solution was developed for the bridge to suit the new track possessions.
Substructure Design
The bridge site is quite close to the Murray River and has a soil profile of over 50 metres depth to rock consisting of alluvial sand and clay deposits of varying density and stiffness. The composition of the rail embankment was unknown but there was some concern that the original timber bridge at the site had been buried in the embankment when the steel bridge had been constructed in the 1920’s. A series of pilot holes were drilled at the tracks to check the pile locations were clear of obstruction prior to the piling commencing.

The other constraints that had to be addressed in the design were that the embankment had to be retained during top down construction with the trains operating on the new bridge and all the piles had to be installed during 2 to 3 hour track occupancies.

If conventional bored cast in place concrete piles were used for the vertical support of the bridge they would have needed to be over 30 metres deep to reach a suitable sandy gravel layer. These piles would have needed to be installed using casing or Bentonite or polymer slurry which was simply not practical without a track possession. Frankipiles were already being used on the project for some of the freeway bridges and Abigroup found the driving of the concrete plug to compact the soils and form an enlarged bulb was very effective and economical in the alluvial soils. More importantly, the speed of installation allowed the piles to be constructed during short track occupancies. Detailed design showed that a 1950kN ultimate vertical capacity could be achieved with a 10m long 600/900 Frankipile and rows of four of these piles were used to support each end of the through girders.

Static load tests to 2975kN (1.5 times design ultimate load) were conducted on 3 Frankipiles at the adjacent freeway bridge over Bridge Street. Dynamic load tests were then conducted on these piles to calibrate the dynamic test results with the static tests. 25% of the piles in the freeway bridge and one pile in each group of 4 piles on the rail bridge were then dynamically tested to confirm their vertical capacity. Frankipile also provided the driving data for all the piles which provided
an assurance that the other piles in the group would perform satisfactorily in service.

A 1350mm deep by 1200mm wide precast post tensioned headstock was used to support the bridge superstructure and distribute the loads to the Frankipile groups. The headstock was connected to the piles with stress bars grouted into piles and anchored into a rebate on the top of the headstock. Given that the headstock had to be stressed to 8 piles of 600mm diameter it was decided to use single 56mm diameter stress bars to connect the headstock to the piles rather than the groups of 2 or 4 stress bars per pile typically used for larger pile foundations. The stress bars were initially installed with the reinforcement cage and poured integral with the pile, however the timeframe available between trains and the tight tolerances on the position of the bars lead to a revised method involving the casting of 127mm ducts into the centre of the piles and grouting the stress bars in during a later track possession.

A 4.2 metre height of the embankment fill had to be retained behind the Frankipiles. Any shoring measures adopted had to be installed during the same short track occupancies as the Frankipiles. Sheet piling, shotcrete with soil nailing and contiguous piled options were all considered. Given the constraints and risks, continuous flight auger (CFA) piles were chosen as the best solution used to provide the restraint of the embankment.

A row of 750mm diameter piles were installed behind the Frankipiles at 1100mm nominal centres. The spacing of the piles was set to miss the rails so piles could be installed without removing the tracks. Wing walls were formed using 6 No. 750mm CFA piles at 1100mm centres. A 1350mm deep by 1050mm wide cast insitu concrete capping beam was then installed over the CFA piles. The lateral loads on the abutment front wall were restrained by frame action between the wing wall piles and the capping beam. The lateral loads on the wing wall were resisted by anchorage of the end piles into the embankment slope and by tie action across the front wall.
In order to install the precast headstock and cast insitu capping over the CFA piles track baulks and temporary shoring were installed so that the track could be lifted out and reinstalled within minutes to maximise the time available for the track possessions.

The passenger services from Melbourne to Albury were terminated at Wodonga for three days and a 40 hour track possession was created on the standard gauge track between Christmas and New Year 2005 to complete the substructure under the tracks. The Frankipiles and CFA piles beyond the standard gaugae track were cut down to the required levels and prepared for the installation of the precast headstock and capping beam reinforcement cage. During the track possession the pile preparation work was completed and the precast headstock was lowered into position onto shim plates then the stress bar anchor plates were installed snug tight and the bearing grouted. Starter bars were installed into couplers on the side of the headstock, the reinforcement cage for the capping beam was lifted into place and then the concrete was cast for the capping beam. The track baulks were then reinstated to span over the abutments with the speed on the first of the trains passing over the works was limited to 20km/hr to minimise vibration on the fresh concrete.

After the track possession the capping beam over the wing walls was constructed and the stress bars in the piles stressed and grouted.

After the bridge superstructure was installed the ground embankment was excavated in front of the abutments and the area between the CFA piles was fitted with strip drains and shotcreted. Brick cladding was installed around the
abutments to hide the piles and provide an aesthetic link to the brick abutments of the old bridge.

**Through Girder Design**

The through girders were sized to be 3000mm deep with a 500mm web thickness to carry the weight of two tracks of 300LA loading and ballast over the 25 metre span. The girder was thickened locally to 800mm wide at the top to reduce the depth of the compression zone to improve the bending capacity and to provide additional lateral stiffness. The girder was also widened locally at its base to provide room for the anchorage of the deck transverse stressing and over the full depth at the ends for the anchorage of the longitudinal stressing. For aesthetic reasons vertical ribs were added to the girder to make it look more like an old steel through girder bridge or the brick parapets on jack arch bridges.

The girders had an overall weight of 140 tonnes and the cost of lifting the girders into position in one length or casting them in one length on the temporary supports for the launch would have added considerable cost to the bridge. The girders were instead cast on the ground adjacent to the bridge in three match cast segments then lifted into position with a 300 tonne crawler crane.

15.2mm strands were used for the post tensioning and the girders contain a pair of straight 22 strand horizontal tendons at the base, two 19 strand parabolic tendons and a 12 strand horizontal tendon in the top flange. The 12 strand tendon was installed to prevent tension and to provide horizontal bending capacity across the joints between segments.

The bridge abutment headstocks were extended in conventionally reinforced cast insitu concrete to form temporary runway beams that were supported on temporary Frankipiles. The ends of the end girder segments were supported on the permanent bearings. Temporary supports using RMD mega-shore tower segments were set up on either side of the roadway to support the other end of these segments and the middle segments. Bracing frames were set up on the runway beams and towers to restrain the segments against lateral movement.

*Figure 4: Through girder segments on temporary supports*
The RTA required the joints between match-cast segments to be coated with epoxy to minimise the risk of water ingress to the tendons. Sealing joints between two precast units is very common but this bridge required three segments weighing over 40 tonnes each to be sealed and stressed together. There was insufficient time to lift the third segment and install the prestress strands before the epoxy would set on the first joint. To overcome this problem, a 50mm gap was left between the girder segments on their temporary supports so that the epoxy could be applied to the faces of the segments with a spatula after they had been lifted into position and the prestressing tendons had been installed. Megapoxy PMES epoxy grout was used on the joints because it has a slower set time that the standard Megapoxy H.

Some initial stress was placed in the two bottom tendons and the top tendon to pull the end segments in to meet the middle segment. This was possible because the ends segments were seated on the permanent sliding pot bearings at the runway beam and a low friction seating on the temporary support tower. Once the construction team was satisfied that the joints had closed on the shear keys, the top tendon and one of the bottom tendons were fully stressed and the other bottom tendon was stressed to 50%. The completion of the stressing of the bottom tendon and the stressing of the two parabolic tendons was undertaken after the deck had been constructed so that there was sufficient weight to balance the prestress.

![Figure 5: Deck Cross Section](image)

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**Deck Design**
A 9.58 metre wide deck was required to accommodate the two tracks at 4.5m centres and 2.5m minimum clearance to the girders with a horizontal radius of 800 metres on the tracks.

As discussed in the background information there was pressure on the designers to keep the deck as shallow as possible in order to minimise the depth the roadway had to be lowered to achieve the required 5.3 metre vertical clearance.
A cast insitu concrete deck was the preferred construction method for minimising the deck thickness by allowing longitudinal distribution of the axle loads and simplifying the connection to the precast through girders. The use of precast concrete was preferred for construction staging because there was insufficient vertical clearance to install falsework under the deck of the operating street which contained a significant volume of heavy vehicle traffic. The soffit of the deck was also required to be close to flush with the girder soffit so depth wasn’t lost with a plank seating.

Precast concrete permanent formwork was used with a composite cast insitu deck as a compromise to these structural and construction staging needs. The formwork units were seated on 150x150x12 galvanised steel angles bolted onto the sides of the girders. The precast units needed to be thick enough to span between the girders with minimal support but be thin enough to accommodate the transverse and longitudinal prestressing. In order to span the 9.58 metres under its self weight a 200mm thick precast formwork unit was required. This unit was too deep to sit the transverse stressing over the top of the unit so the decision made to adopt this thickness and cast the transverse ducts into the precast units. The cast insitu slab depth was set at 325mm to provide an overall deck depth of 525mm.

The formwork units were supplied in 2300mm sections contain 3, 12 strand tendons at 770mm centres. The deck units included shear stirrups to ensure composite action between the precast and cast insitu concrete and the shear capacity of the deck. The deck units were manufactured with a 75mm precamber that deflected down to 10mm precamber after installation. The units were fitted with a pair of tie roads at midspan that extended up to temporary steel cross beams to support the weight of the cast insitu concrete until it gained strength and was stressed.

The cast insitu deck was reinforced with light top reinforcement throughout and longitudinal reinforcement in the bottom across the joints between the transverse spanning precast units. The deck was prestressed with 5, 12 strand horizontal longitudinal tendons at mid depth to provide compression and longitudinal bending strength. 19 strand tendons were also provided adjacent to each girder to provide additional stress and strength to the girder. The profile on the transverse tendons and the 19 strand tendons required the deck to be thickened locally from 525mm to 700mm adjacent to the girders.

The 12mm strand tendons at 770mm centres were sufficient to provide the strength and maintain compression on the joint between the deck and the girder for all but the ends of the deck where 6N32 couplers were provided top and bottom for some conventional reinforcement to provide additional strength where there is a concentration of loading on the edge of the deck.
Superstructure Installation

The completed superstructure needed to be lifted off the bearings and placed on skates to be slid across into position. The skates travelled on rails made from 200PFC that were laid flat with the vertical flanges preventing the skates from deviating off course during the slide. The girder had to be long enough and the runway beam wide enough for these rails to pass either side of the bearing bottom attachment plate. The weight of the superstructure was just over 600 tonnes and there was insufficient strength in the deck to lift the superstructure off the bearings with a single jack on the inside of the bearing. The girder therefore also had to be wide enough to position the jacks either side of the bearing to lift the superstructure. Corbels were cast on the outside of the girders locally at the abutments to allow sufficient room for the installation of the jack and skates.

The superstructure was completed a week before the Easter track possession and was prepared for the installation with the temporary support structures being removed and the ballast mat and new sections of track installed on the bridge. The broad gauge track was closed prior to the possession and the girders and transoms for this track along with the disused track were lifted out and the abutment brickwork cut down below the new girder level. The bridge was slid across into position using hydraulic jacks anchored to the runway beams between the rails.

During the track possession, the final section of the existing bridge was lifted out and the abutment walls cut down and bridge approaches cut down. The area behind the new abutments was also prepared for new 4m long precast concrete approach slabs. The rails were extended across the permanent headstock and the bearing bottom attachment plates were installed in their final locations.

Figure 7: Superstructure Slide
The bridge was slid across into position then lowered down and the bearings bolted to the bottom attachment plates. On one side of the bridge slipped guided bearings were used with restraint in the transverse direction. On the other side free float pot bearings were used. 450x600x102 laminated elastomeric bearings were placed between the deck and the abutment curtain wall to restrain movement in the longitudinal direction. This was done so pot bearings were not loaded up from any movement of the abutments from earth pressure longitudinal train collision loads.

The approach slabs were installed, the track reinstated and the track reopened 8 hours prior to the end of the possession.

Conclusions
Sliding concrete through girder rail underbridges into position during track possessions is a common technique. This bridge did however contain some innovative solutions to address time and rail and road occupancy constraints.

The use of Frankipiles and CFA piles with a combination of precast and cast insitu in the abutments overcame the problem of limited track possessions and difficult geotechnical conditions. The use of precast concrete segments for the through girders and incorporating the transverse prestressing into thicker formwork panels overcame problems of cranage and road occupancy.

The success of the bridge structure was a breakthrough for the team, stemming from collaboration of design and construction during all phases of the project, fostering innovation, and resulting in a satisfied client.

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