

DESIGN OF INTEGRAL STEEL LADDER DECK BRIDGES IN NEW ZEALAND – A CASE STUDY OF GILCHRIST STREET BRIDGE

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ABSTRACT

The Te Rapa bypass project is a 7.2-kilometre section of the Waikato Expressway, which was completed in 2012. The road diverts SH1 traffic away from the busy commercial area of Te Rapa, north of Hamilton. The landmark structure of the project, Gilchrist Street Bridge, is an elegant steel-composite structure that carries two lanes of state highway traffic over the Gilchrist Street Interchange. The steel-composite structure is an efficient and optimised design that was developed by the project team as an alternative to the specimen design.

The new bridge is a 60 metre 2-span integral structure, comprising of composite concrete-steel ladder deck construction with piled foundations at a skew angle of 24 degrees. This bridge is in a seismic region with poor ground conditions. The light weight superstructure allowed a slender bridge, reducing both the visual impact and cost over the specimen design. Key benefits include the removal of pier diaphragms, providing small diameter columns and minimising the foundation works.

The final solution is a magnificent example of elegant bridge engineering and demonstrates what steel bridges can offer to New Zealand - a combination of reduced structural weight, durability and aesthetics. The light weight steel superstructure minimised ground works and is detailed to minimise whole of life costs by achieving maximum durability.

Introduction

In 2008, the New Zealand Transport Agency (NZTA) identified seven Roads of National Significance (RoNS) which are key to enabling economic growth. These roads focus on improving safety and journey reliability, and reducing journey times between main population centres. The NZ\$1 billion Waikato Expressway is one of these RoNS, which will provide a divided four-lane highway between Auckland and Cambridge by 2020.

The construction of Waikato Expressway has been divided into a number of sections (see figure 1). The Te Rapa section is a spur, connecting the Ngaruawahia section with north west Hamilton.

Te Rapa section

The 7.3km route of the Te Rapa section takes traffic away from the industrial/commercial area of Te Rapa Park, and eliminates the need for state highway traffic to travel through roundabout and signalised intersections. The route connects to the local roads via grade separated interchanges at Horotiu, Central Road and Gilchrist Street. The northern end of the route is to be connected to the future Ngaruawahia section of the Waikato Expressway (see figure 2).

The existing State Highway 1 route takes traffic from Horotiu, through the industrial/commercial area of Te Rapa Park, passes the newly developed The Base shopping complex, along Avalon Drive and connects to the recently constructed Avalon Drive Bypass.

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Figure 1 Waikato Expressway (courtesy of NZTA)

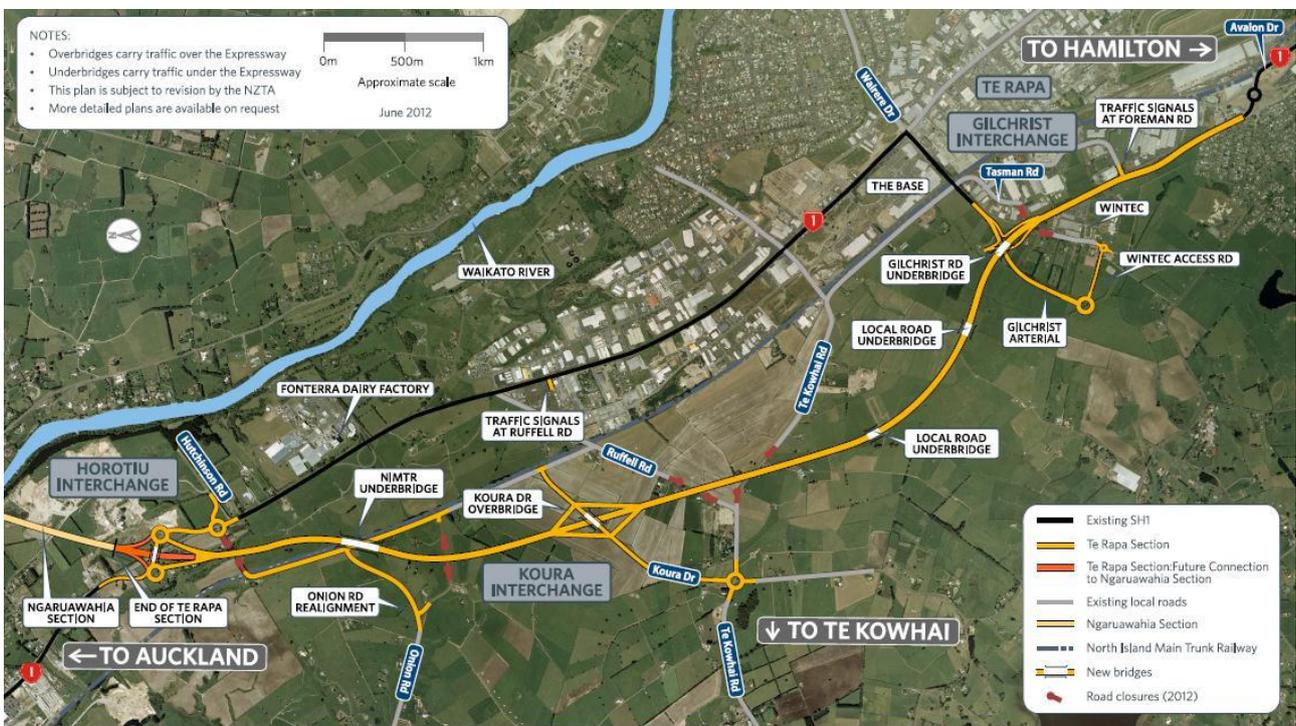


Figure 2 Te Rapa Route (courtesy of NZTA)

Major bridge structures for the Te Rapa Project include the SH1 Horotiu Interchange Bridge, NIMTR Bridge, Central Interchange Bridge, Gilchrist Street Bridge and two local road underbridges. The NIMTR Bridge and Gilchrist Street Bridge are of steel-concrete composite construction, and other bridges are precast prestressed concrete beams.

The Te Rapa Project has been procured through a competitive alliance process for NZTA, a New Zealand government agency. The Te Rapa Alliance is the successful tenderer for the NZ\$210 million project, set up by NZTA, consultants Opus International Consultants and constructor Fulton Hogan.

Tender design

The key drivers from the Principal's Requirements significantly influencing the structural form were as follows:

- Vertical Clearance of 5.1m – the local roads (Gilchrist Street and the re-aligned interchange) only required minimum clearances, but given this area of the job is in an embankment fill zone, the structural depth needed to be minimised to reduce fill quantities.
- Spill through abutments – due to the highly visible location, the abutments were required to be sloped at a gradient of 2H:1V

The Alliance partners worked collaboratively to develop numerous options which were tested against the PRs. For each option the costing team produced comparative costs identifying cost critical items. This allowed the design team to concentrate on high value items, such as the pier caps, piling, construction depth, overall bridge length and Whole of Life Cycle costing.

The ground conditions at the site comprised 10-12m of interbedded loose to medium dense sands and silts, 20-22m of medium dense sands and silts/sandy silts. This necessitated a piled solution. With driven piles widely adopted across the rest of the bridges on the project, the options were limited to 610mm bottom driven steel shells and 450x450mm top driven precast concrete. A project wide decision was taken to use the 610mm bottom driven shells.

With the construction depth and overall bridge length identified as cost critical items the team focused on a developing an innovative roading alignment that reduced both. The team developed an alternate interchange arrangement for Avalon Drive, Gilchrist Street and SH1. This alignment reduced the overall length of the bridge to 62m by eliminating the back spans and providing a single centrally placed pier. Additional benefits to this arrangement included a reduced construction depth with the knock on effect of minimising the fill embankment.

With the alternate intersection arrangement addressing the bridge length and construction depth, the focus of the team changed to minimising the substructure requirements. Minimising the structural dead load was a key consideration and light weight steel composite options provided significant substructure savings. The options considered are given in Table 1.

The whole of life costing of the bridge was minimised through eliminating the expansions joints and removing the bearings at the abutments by adopting integral construction. The bearings were retained at the piers as the cost and Health and Safety implications of constructing an integral pier cap did not offer the required whole of life savings to make it a viable option.

Table 1 Tender Design Options

Option	Span	Articulation	Superstructure Form	Substructure Form	Structural Depth (m)
Specimen Design	110m (26+35+28+21m)	Simply Supported on elastomeric bearings	Prestressed composite Super-T beams	Piled abutment bankseat, piled pier with single column and pier caps.	1.70
Option 1	62m	Integral	Composite steel beam and slab (6N° beams)	Piled abutment bankseat.	2.1m
Option 2	62m	Integral	Composite steel beam and slab (8N° Beams)	Piled abutment bankseat.	1.7m
Option 3	69.8m (13+43.8+13m)	Integral abutments, Continuous at Piers	Steel Ladder deck (2 Beam), with composite concrete deck	Piled abutment bankseat, piled pier with 900dia column	2.1m

Option	Span	Articulation	Superstructure Form	Substructure Form	Structural Depth (m)
Option 4	69.8m (13+43.8+13m)	Integral abutments, Continuous at Piers	Steel Ladder deck (3 Beam), with composite concrete deck	Piled abutment bankseat, piled pier with 900dia column	1.8m
Option 5	69.8m (13+43.8+13m)	Integral abutments, Continuous at Piers	Composite steel beam and slab (6N° beams)	Piled abutment bankseat. Piled piers with pier cap and columns	1.35m
Option 6	61.6m (30.8+30.8m)	Integral abutments continuous at piers	Steel ladder deck (2 beams) with composite concrete deck.	Piled abutment bankseat, piled pier with 900dia column	1.8m

Option 6 was selected by the Alliance as it achieved all the Principals Requirements at the lowest whole of life and construction costs. The design team adopted safety through design principals in the tender design to ensure that the ongoing operation and maintenance of the bridge could be undertaken as safely as possible. An example of this is the increased pier head, which provides for jacking points to allow the replacement of the elastomeric bearings which have an expected service life of 40 years against the bridge design life of 100 years.

Detailed design

Structure summary

The final structural form comprised a two span 59m long ladder deck steel superstructure with a constant steel plate girder beam 1.5m deep. The two spans of approximately 29.5m cross the Gilchrist Interchange (refer to elevation in figure 3). The bridge is founded on 610mm diameter steel encased bottom driven concrete filled steel shell piles, with 7 piles per abutment and 8 piles per pier. The spill through batter slopes were kept at 2H:1V for slope stability, and to meet the PRs. Given the overall road is in fill embankment in this section, minimising structural depth was a key aspect of the design as discussed above. A discussion on the aesthetic aspects of the design is covered in the following section.

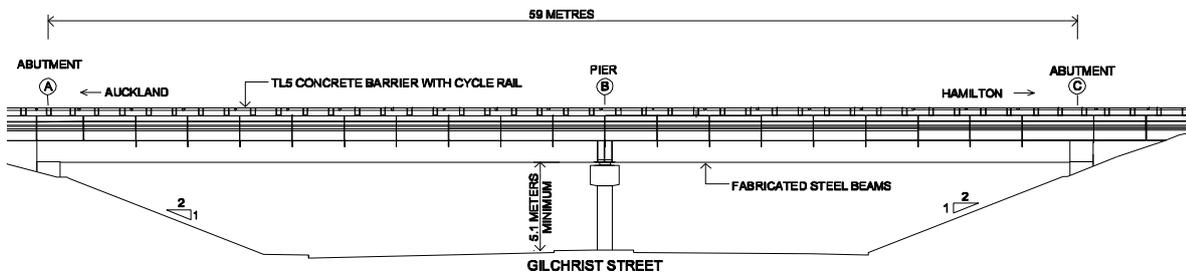


Figure 3 Elevation of Gilchrist Street Bridge

Superstructure design and erection

The superstructure was designed considering ease of fabrication and available steel plate sizes. This information is readily available from Heavy Engineering Research Association website (hera.org.nz). This influenced the choice of plate sizes on the steel plate girders for the superstructure and discussions were held with the construction team at an early stage on how the construction sequence would influence design. The decision was made to have welded splices in the shop to the maximum transportable length (given the relatively easy access to site), to minimise site works. Bolted splices were used at points of contraflexure either side of the pier with plate size changes optimised based around available plate lengths. No limitations were given in terms of crane size or access ability, and given the relatively lightweight steel superstructure of approx. 170t a three stage process was envisaged for steelwork erection, involving building the structure on the ground in three lengths and lifting into place. This was intended to minimise work at height and allow any issues to be solved at ground level where there was a large lay down area.

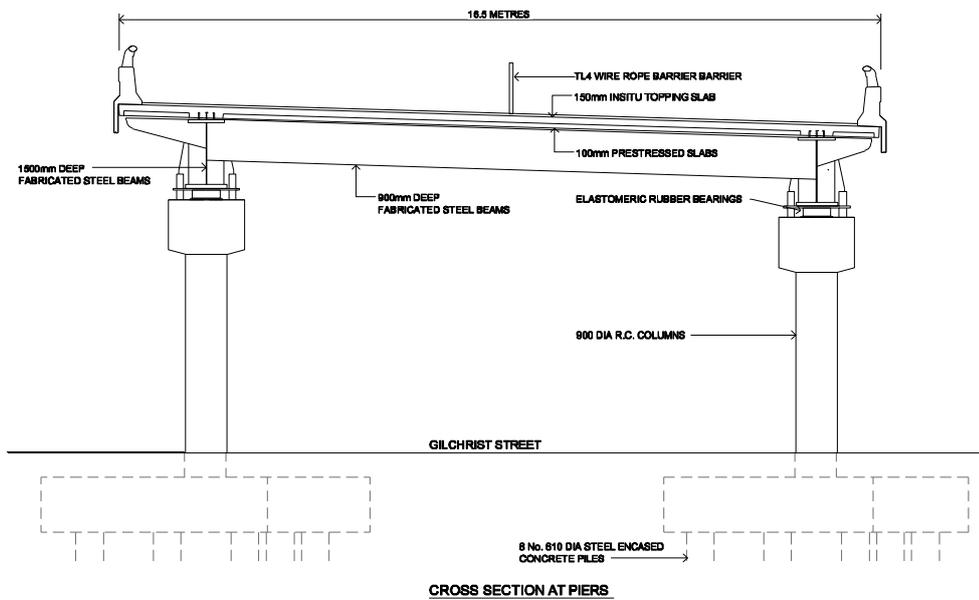


Figure 4 Cross Section of Gilchrist Street Bridge

The concrete deck was formed using a series of precast prestressed planks spanning between the transverse girders (see figure 4) as permanent formwork. The surface of the planks was roughened with a type B construction joint finish (in accordance with NZS3109) and spirals were cast in to provide positive connection. Shear studs on the beams (both longitudinal and transverse) provided composite action. All reinforcement design and shear stud spacing was based on a multiple of 150mm spacing to limit potential clashes on site, and aid constructability.

Integral connection

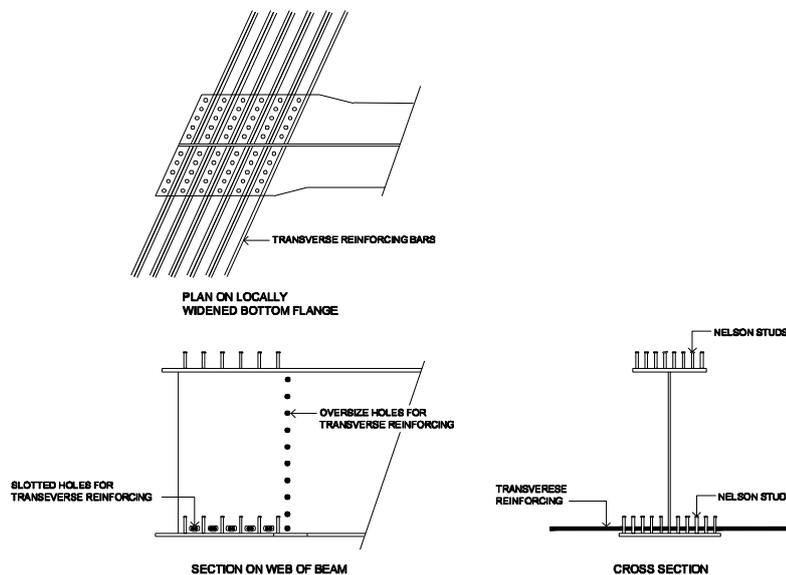


Figure 5 Detail of Integral Connection

One of the most challenging aspects of the design was the detailing of the integral connection from the steel beams to the concrete abutments. Typical examples for detailing of this connection are often taken from European guidance standards (such as the Steel Construction Institute Guides (SCI)), but these details need adaptation for New Zealand design, due to the reversing moments during a seismic event. A number of options were considered for the connection, but the final solution used a series of shear studs welded to the flanges to transfer forces to try to reduce some reinforcement congestion (refer Figure 5). Holes through the web were oversized to allow positive connection of the beam with the reinforcing cage and the first plate girder segment had an increased web thickness to account for the loss of shear area.

Safe by Design

Safety during construction was an important consideration throughout, and where possible the design was modified to suit working procedures. For example, typically a pier head might be required at the central pier but this was designed out to reduce working at height (this also benefits the bridge aesthetics). The cross section through the bridge (see figure 4) shows permanent cantilevers were used rather than a temporary formwork system, which prevented stripping of the formwork at height.

Aesthetics and Urban Design

During the tender design, bridge architects and urban designers were part of the design team, and there was a strong focus on the aesthetic of bridges on the project. Each bridge was assigned an aesthetic performance level in the PRs, based on its location and impact on the surroundings. Although initially Gilchrist had a level two aesthetic rating, the design team quickly identified that it was a highly visible structure being at the Southern entrance to the new bypass, and adjacent to the Waikato Institute of Technology (Wintec).

Consideration of the visual impact of the bridge was thus a priority for the designers from the start. Using steel gave the team an optimal platform for delivering on this aesthetic priority with a slender superstructure, slender columns and the removal of the pier head. The beams were maintained at constant depth, with the increasing design loads across the structure taken by changes in plate thickness. The use of outer stiffeners was also limited to provide clean lines and help maintain a slender appearance.

One subjective aspect of the design was the use of relatively deep cantilever elements. The initial aesthetic preference of a temporary formwork system for the cantilevers was rejected on health and safety and cost grounds. This led to the required permanent steel cantilevers. A slender short cantilever was initially proposed but the cantilevers were lengthened by the designers and increased in depth to help reduce the actions on the transverse members. This reduced the overall steelwork quantities at the expense of a more robust cantilever “rib” which is visible on the elevation of the bridge (see figure 10).

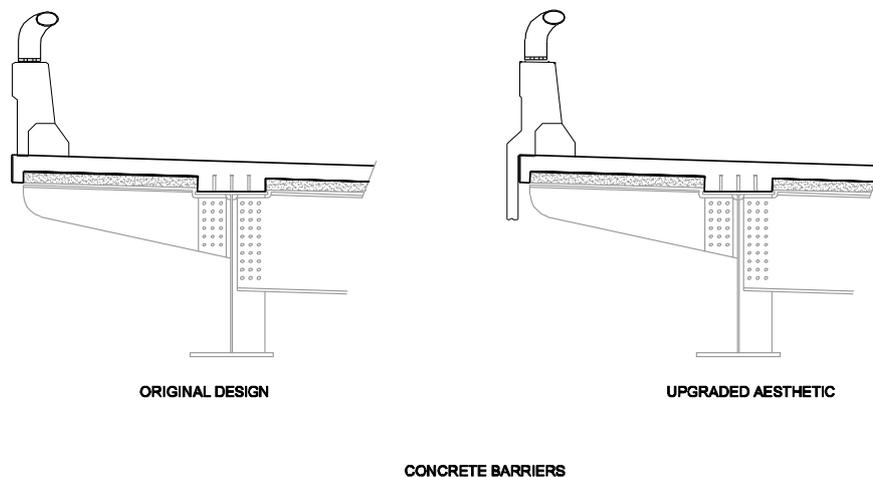


Figure 6 Parapet before and after aesthetic treatment

The collaborative approach of the design and construction teams allowed for further upgrades in the aesthetics for the bridge when the status of the bridge was upgraded by the Client during design to a level three structure. Particularly, the focus of the team was to hide any onsite construction joints, and a teardrop downstand was added to the parapet (See figure 6). The downstand particularly aids the clean lines achieved by the final solution, the length sized to ensure shadow lines on the steel beams reducing the feeling of depth and enhanced the slender form (refer figure 7, 10).



Figure 7 Bridge at night

The spill through abutments also received urban design treatment, with a flax pattern added in paving stone and under bridge lighting used to enhance the visual appeal at night and increase safety. Particularly the under bridge lighting effects were integrated well into the structural design, with the ducting carefully hidden under the bridge, providing no visual impact on the elevation (figure 7). The road lighting posts on the bypass were sited on the abutments of the bridge, the length maximised through collaboration between lighting and structural teams to keep the constant depth of the structure and minimise visual intrusion.

Whole of Life considerations

One criticism of steel bridges is the future cost of maintenance and the fluctuation in price of steel. Although this paper does not cover the procurement risk, it is worth noting that given issues with piling during construction, a concrete solution at this bridge would have carried very significant cost penalties over the steel solution. Therefore, the designers recommend that when high risk ground conditions exist, perhaps where a specific founding depth is being targeted, that the lightweight superstructure provided by steel can help to mitigate this risk.

Throughout the design, consideration of the future maintenance costs were a high priority across the structures. Concrete aspects of the structure were detailed for a 100 year design life, with focus on the interface between steel and concrete elements. At the integral connections and on the top flanges of the main structural elements the paint details in HERA Report R4-133 section 12 (El Sarraf, 2011) were used to ensure maximum design life. The paint system chosen was inorganic zinc silicate (IZS2) and is expected to deliver a time to first maintenance of 25 years for all structural members.

The use of a steel ladder deck allowed the designers to minimise the number of bearings on the structure, with only two at the central piers. This minimises the maintenance requirements for inspection and replacement. A widened pier head was provided to allow jacking and replacement under service load.

The benefits of integral construction for whole of life costs are well documented elsewhere, and the same benefits are applicable here: the removal of bearings, bearing shelves and movement joints. Additionally, in a seismic region the integral connection provides additional robustness and removes the need to form large transverse shear keys or install linkage bolts. To provide this robustness a pier cap or continuous pier pilecap would have otherwise been necessary.

Construction

The construction process in an Alliance type contract provides a different type of environment which fosters and develops innovation to provide savings in time and cost for the overall project. This was brought into sharp focus early in the construction programme, when ground risk eventuated, resulting in a reduction in pile end bearing capacity of almost half. This required the provision of almost double the number of piles to carry the additional vertical load, and the teams provided a staged solution in three weeks (redesign to construction).

Piling issues

To take advantage of the higher strength reduction factor allowed under the design code, the geotechnical team specified the use of pile driving analysis (PDA) testing during construction to verify the predicted vertical pile capacity. During construction, the PDA results came back at almost half of the predicted vertical capacity. Increasing the length of the pile was deemed unfeasible as the depth of the denser ground material was estimated to be at 40m below ground level. This would require an increase in depth of almost double the designed length. Instead, the number of piles at the pier and abutments was increased.

This was relatively straight forward to achieve at the pier, as each column is supported on individual pile cap. The redesign had to ensure the spacing of the piles was sufficient to prevent any differential settlement. It was more complex at the abutments. Due to the nature of the two beam ladder deck type bridges, much of the construction loading is concentrated on the outer edge of the abutments. To prevent having to widen the abutment the team proposed a solution which took into account deflection of the piles under load prior to casting the integral connection and allowed some redistribution of load by the stiff abutment cap. This method was then confirmed by both the geotechnical and structural peer reviewers.

The additional piles in both the abutments and piers had been detailed to resist gravity only by providing only 50m steel casing embedment into concrete and a centrally placed reinforcing bar. This connection is designed to transfer shear only. Timescales did not allow the designers to fully take into account any increases in lateral resistance that may be provided by a moment connection at this point.

Steelwork Erection

All the main girders, cross girders and outriggers were labelled and transported from the steelwork subcontractors' fabrication yard in Auckland to site over three days. Sections of the main girders were then assembled together on the ground. Bolts were all friction grip type connections and were tightened using a hydraulic bolt tightening machine in accordance with the part turn method as per NZS3404.



Figure 8 Photos of steelwork erection

The erection of steelwork was then carried out with one 100 tonnes crane (figure 8), two cherry pickers and seven personnel over two weeks. This was in excess of what the designers had intended. Instead of building

the structure on the ground and lifting into place as per designed, the contractor opted for a smaller size crane and to lift the steelwork in beam by beam to save on crane costs.

The detailing of the assumed sequence worked well for most of the revised sequence, but there were some issues at the piers and the abutments. Due to the presence of bearing stiffeners along the main girders on both sides at the pier, there was not enough space to rotate the cross girders in place, so minor modifications had to be made to the design. In the future, detailing at bearing stiffeners should always use double plates at bolted connections to add flexibility in erection.

The biggest challenge was getting the end trimmer beams in place. These were designed to be lifted in with the main beams to provide stability at the reaction points during lifting and until the integral abutment was cast. Due to the skew angle of the bridge, the beam clashed with both the end bolted connections and the abutment starter bars. This provided a significant delay and required redesign to allow modifications to the elements to allow the erection to be completed.

The corrosion protection of the beams damaged during erection were repaired as per recommendations in HERA report R4-133:2011 (El Sarraf, 2011) in order to ensure minimum 25 years to first maintenance is achieved.

Composite Deck Construction

The deck construction was relatively straightforward. The predominant use of precast panels as permanent formwork substantially reduced the need for timely erection and stripping of formwork. However, strict quality control was required to ensure that four layers of reinforcing fit into the top 150mm thick cast in-situ deck, while still achieving minimum spacing between layers and the required cover. In a deck of this type, for construction ease a 275mm overall depth deck, an optimised formwork system with a reduced depth of less than 75mm or the use of only three layers of reinforcing with careful consideration of shrinkage and thermal stresses should be considered.

Cast in inserts provided in the precast slab for hanging the aesthetic lighting system worked well and lined up as per design. This is considerably easier than post fixing these support systems.

Parapet Construction

The designers opted for precast parapet to the bridge deck through a cast in-situ stitch joint. The in-situ joint was poured using self-compacting concrete to eliminate the need for vibration. The advantage of precast parapet is shorter construction time, higher quality finish and avoided the need for complicated formwork on site, especially with the parapet downstand design.

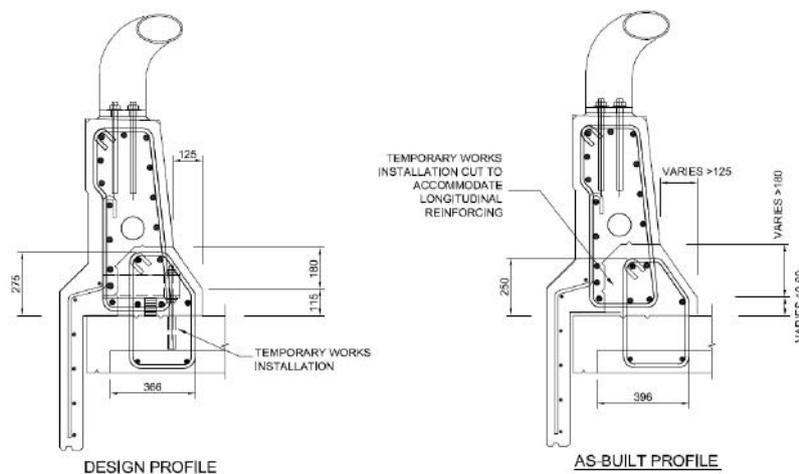


Figure 9 Showing clashes with bars from slab, barrier

During construction, the parapet starter bars (which were cast into the permanent formwork – see figure 9) were out of position in plan. Although the designers had provided a cast in levelling strut to prevent the need for onsite propping, the starter bars clashed with these struts and they had to be removed. To retain cover for

durability, the profile of the barrier had to be modified slightly (but still within the F-type barrier shape). It is clear that when using precast barriers and precast permanent formwork, significant co-ordination is required to reduce the potential for clashes. The contractor needs to ensure that tight tolerances are retained through the production and construction process.

Conclusion

The positive design, construction and aesthetic outcomes of Gilchrist Street were possible through the use of steel as the primary superstructure material. The slender and clean superstructure form is fitting for the highest aesthetic bridge on the Te Rapa Bypass. At the Southern End of the bypass, with significant traffic both over and under, the light steel deck allowed slender pier columns to be used minimising the visual impact. Through collaboration between the design and construction teams, additional detailing such as under bridge lighting and barrier treatments further enhanced the positive environment created (figure 10).



Figure 10 Bridge at completion

The use of a steel as a construction material helped to significantly reduce foundation costs, which was brought in to sharp focus when unexpected ground conditions were encountered during construction. The lightweight superstructure meant that relatively minimal additional costs were incurred to achieve design performance.

The whole of life costs of the bridge were considered throughout, with 25 years to first maintenance for the steel work. The bridge is also integral at the abutment which provides the long-term durability benefits of this type of construction. A potentially costly and complex diaphragm at the piers was also removed by the use of steel as a construction material.

Acknowledgments

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El Sarraf, R and Clifton, 2011, *New Zealand Steelwork Corrosion and Coatings Guide*, New Zealand Heavy Engineering Research Association (HERA), Auckland