SPANNING TO THE FUTURE – STRUCTURED IN STEEL

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The Steel Bridge Group (SBG) is a technical forum that has been established to consider matters of high priority interest to the steel bridge construction industry in the UK and to suggest strategies for improving the use of steel in bridgework.

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Front cover:

The Jack Williams Gateway Bridge in Wales
courtesy of Costain

New Pooley Bridge in Penrith, Cumbria
courtesy of Robert Mark Lawson

SPANNING TO THE FUTURE – STRUCTURED IN STEEL

FOREWORD

Even in the Digital Age, modern day society relies upon physical connections, and bridges play a critical part in supporting transport networks of all types. The greater the degree of connectivity, the stronger the human connections that are made and these are vital not only for national economies but for all aspects of our everyday lives. Often this connectivity goes unnoticed, and it is only when a bridge is unavailable that its true value is understood. The need to maintain our structures is just as important as the desire to continue building anew.

But it is increasingly obvious that we must be far more considerate with the Planet and our effects upon it, as transport infrastructure and the movement it facilitates is a significant contributor of carbon emissions. The challenge of the Climate Emergency demands that designers, especially in the most prosperous counties, must act to reverse the effects of global warming if we are to protect the prospects of future generations.

We must consume less and design with care, and this means using materials efficiently and to greatest effect. We must encourage modal shift to low and zero carbon modes of transport and we must question the need for projects at all stages.

The modernist mantra “Less is more” still applies – lightweight is good and steel can be excellent in this respect – but we must also seek to do more with less, meaning reimagining structures to serve for longer and in different ways, to allow them to be repurposed and reused before they are recycled. We must increase durability and the resilience of structures to extremes of weather and changes in traffic use.

If we are to build, there are many benefits of using steel as a construction material. The following Case Studies illustrate creative collaborations between engineers, architects, contractors and their clients in service of society.

Martin Knight FRIBA FICE Hon FIStructE
Founding Director, Knight Architects

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INTRODUCTION

The use of steel in bridges has a number of advantages such as enabling slender forms due to the excellent strength / weight ratio of the material, ease and speed of construction and superior aesthetics to name a few. Although the above advantages are quite obvious, there are numerous others that are sometimes overlooked by planners during the early stages of a project's development.

One such benefit is very good durability (when a bridge is properly designed and detailed) leading to low whole life costs, and the potential to recover and recycle material at the end of the working life of the structure.

One of the misconceptions about steel bridges is that the steel must be coated and this coating will have to be replaced every 20 years, typically. This is because of the exposure to a wet environment and due to degradation of the coating due to UV radiation from exposure to the sun. However, proper detailing and the use of modern products are likely to significantly extend the period before replacement. Recent advances in coatings' research mean that there are products available that can lead to a prolonged life, and they are also more sustainable (not only due to lower carbon associated with manufacturing, but also due to the reduced maintenance and prolonged life of the products). Some fluoropolymer paints, for example, can last significantly longer (reportedly up to 60 years, although that depends on conditions) before they require maintenance or replacement.

Furthermore, certain types of steel do not require any protection. Weathering steel is often specified in bridge projects and has an increased resistance to atmospheric corrosion and a distinctive appearance due to its natural 'rusty' colour. A dense layer of oxides (rust patina) forms on its surface, slowing down the corrosion process and protecting the steel surface. As a result, weathering steel does not require paint coatings or other additional corrosion protection. The patina has the ability to self-renew its protective layer, which also makes it resistant to minor surface damage during transport, installation and operation. In addition, the material does not normally require additional treatment during the life cycle of the structure.

The first applications of stainless steel in bridges have also emerged in the last few years. Duplex stainless steel grades have a ferritic-austenitic structure and they combine high mechanical strength and excellent corrosion resistance which make them a competitive choice for bridges. Their superior mechanical strength and corrosion resistance make them an ideal choice for certain bridges, especially in coastal environments or where high levels of de-icing salt are used. The higher initial cost (over carbon steel) is often compensated by the much lower whole life costs in such situations.

Another misconception is that steel is not an environmentally friendly material. This is understandable and contains some truth in absolute terms, because, as with any man-made material, steel production is associated with a carbon footprint. However, a holistic approach involving all stages of a structure’s life-cycle (cradle-to-grave assessment) is often not considered when comparing steel with other materials. Also, the difference in methods (process routes) associated with steel production may not be fully taken into account when making comparisons. Structural steel can be almost fully recovered and recycled (typically more than 95%), and a large percentage of new steel is produced by using scrap material. Recent trends in relation to steel manufacturing involve the development of ‘green’ steel, i.e. steel produced almost entirely from scrap (recycled) material and by using renewable sources of energy to power the production, thus reducing the carbon footprint of the structure.

The case studies that follow are examples of steel being used effectively and efficiently in bridge construction, to exploit the benefits identified above. It is hoped that they will inspire and inform planners and designers to always consider the use of steel during the initial planning phase of a project, and to use it when it is appropriate and in an appropriate way.

JACK WILLIAMS GATEWAY BRIDGE

2.1 Facts and figures

Client:	Welsh Government
Consulting Engineer:	AtkinsRéalis
Architect:	Knight Architects
Main Contractor:	Costain
Steelwork Subcontractor:	Victor Buyck Steel Construction
Construction Commenced:	September 2014
Public Opening:	October 2018
Steel Tonnage:	1650 tonnes
Protective Treatment:	Deck: Weathering Steel Arch: Type II to 1900 Series
Further Information:	https://www.atkinsglobal.com

Figure 2.1
The Jack Williams Gateway Bridge

courtesy of Costain

2.2 Design basis

Design Standards:	BS EN 1990 to 1994 and 1997 Design Manual for Roads and Bridges[1] as implemented by Welsh Government Manual of Contract Documents for Highway Works[2]
Loading:	Load Model 1 and Load Model 3 using SV196
Design Life:	120 years

2.3 Location

A 46 km stretch of the A465 from Abergavenny to Hirwuan is steadily being improved by the Welsh Government to dual carriageway standard, as it is critical to the social and economic regeneration of the Heads of the Valleys region. It also provides resilience as a diversion route for the M4 corridor through South Wales. The Heads of the Valleys Road Improvement project was divided into six sections for development, with the 7 km long Section 2, running from Gilwern to Brynmawr, climbing through the steep terrain of the Clydach Gorge. There are three grade separated junctions including the 'staggered dumbbell' type interchange at Brynmawr. The Jack Williams Gateway Bridge carries the road linking the two junction roundabouts, over the A465 at a height of 25 m above the valley floor.

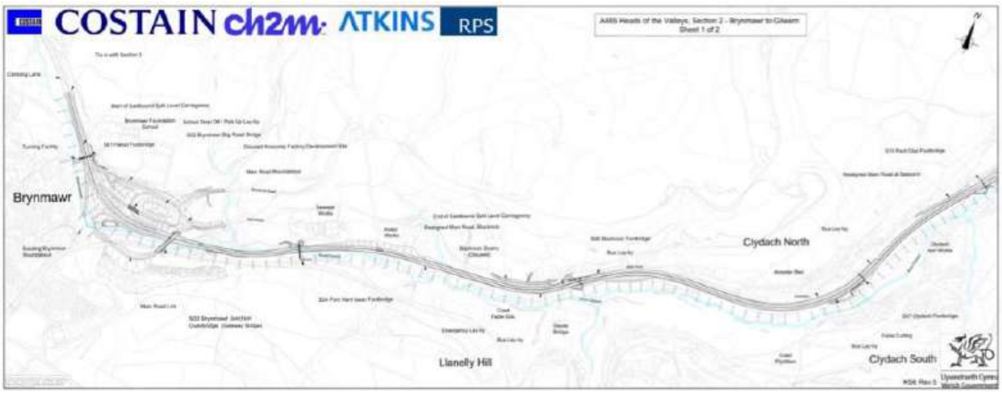


Figure 2.2
Location plan

courtesy of Costain

2.4 Bridge context

The Welsh Government had previously developed the preliminary design of the whole 46 km of the route with the A465 Heads of the Valleys Road Landscape Strategy, a route wide objective. The strategy included proposals that the sequence of experience for road users along the scheme was marked by transition zones and gateways. The transition from the steep sided gorge to open high plateau at Brynmawr was an obvious location for a gateway and the preliminary design had an example gateway bridge structure carrying the main road over the junction.

The contract was procured through Early Contractor Involvement procurement, for the development of the preliminary stage through publication of draft orders, Public Local Inquiry and subsequent construction. Fundamental to the tender design was a drive for improved cut/fill balance. The Brynmawr junction layout was altered from the specimen design, in which it was constructed online to the existing road, to an offline design to the north of the existing junction with a split-level carriageway for eastbound and westbound roads. The alternative design link road that connects the junction dumbbell roundabouts crosses over; the A465 dual carriageway, a geological Site of Special Scientific Interest (SSSI) and two Scheduled Ancient Monuments (SAM).

The Clydach Coal Levels (SAM) are the remains of tunnels for the River Clydach, mining adits and connection to the Clydach Tramroad (SAM) developed in the 18th century to exploit coal and provide a transport route to what is now the Brecon and Monmouthshire Canal, at Gilwern. The new road represents the 21st century development of that originally constructed transport corridor. The Clydach Tramroad is now part of National Cycle Route 46.

2.5 Bridge conceptual design

Two tender designs were proposed, a conventional bridge and a gateway feature bridge. The conventional bridge was a three-span structure with the largest 65 m span at the north end and with one support located in the SSSI. As a result of the 65 m span over the A465, the bridge would have adopted a weathering steel concrete composite bridge deck. The alternative gateway bridge was designed to avoid any foundations in the SSSI. To bridge the 120 m gap cable stay, truss and arch forms were considered, with the arch having clear advantages over the other options in terms of cost and appearance in the landscape.

The bridge was conceived as a structure that would be more than just the basic connection of two sides of a junction. The social imperative that had been recognised by the Welsh Government and captured in the 1999 Head of the Valleys Landscape Strategy, required a gateway bridge that would be dramatic, inspirational and create a sense of place. This was fulfilled with the spectacular steel arch of the Jack Williams Gateway Bridge. As you drive up the sweeping Clydach Gorge, the bridge is seen initially through tantalising glimpses; the full visual impact being revealed as you arrive at Brynmawr with the arch dramatically spanning the whole valley as it springs from the steep sided gorge.

The detailed form of the arch then emerged from consideration of the steep gorge and the perception of the road user. Approaching from the west, the junction has a large left hand curve which gives a constantly changing view of the bridge. When viewed from an angle, bridges with two parallel arch ribs present competing views where the ribs are not aligned. The link road was also highly skewed across the valley so the skew span was longer. An arch rib below the bridge deck was not feasible owing to lack of headroom over the split-level carriageway. The resolution to these constraints was a single arch rib that spanned the valley roughly square to the sides of the gorge and bridged the link road from one side to the opposite corner. This form of skew single arch rib had been built in the UK before, notably at Hulme in Manchester and the Clyde Arc in Glasgow, but at 118 m span Jack Williams is the largest bridge of its type in the UK and the first time that the skew arch typology has exploited the local topography to shorten the main span.

The tender concept design was extremely well received by the client's project manager who could see immediately that this bridge would not only provide the gateway structure required but would also create a notable and inspiring landmark feature for the project and the local community.

The concept design was developed during the preliminary design stage with Knight Architects providing valuable improvement to shape the arch over the headroom constraints, determine abutment and arch springing layouts and consider materiality. The link road alignment was refined to make it straight, removing complexity from the bridge design, but requiring a deeper highway cutting to the south of the bridge. The arch rib was also centralised over the link road so that the hanger layout was symmetric and vehicle clearances improved. The distinctive rise of the arch, cresting 25 m above the link road and 50 m above the valley floor, was dictated by ensuring the hangers were at least 5.7 m above the carriageway in order to avoid the risk of vehicle collision. The H1 category vehicle restraint system was set in board at the edge of the 7.3 m wide carriageway. The pedestrian walkway and cycleway was positioned on the east edge of the bridge where the best views down the Clydach Gorge were to be obtained.

Although a concrete post tensioned arch rib could have been constructed this would have required substantial temporary works to restrain the cantilever construction from each side and a separate set of temporary works in the SSSI and over the A465 to form the deck. A steel box girder arch with a steel concrete composite deck would be cheaper, less intrusive on the SSSI, more compact in cross section and require simpler temporary works. Construction would also be less prone to disruption from the significant risk of severe weather.

2.6 Design details

The bridge is constructed from S355 grade weathering steel so that future maintenance is minimised, but after extensive discussion, it was chosen to paint the arch rib in a light colour, as this would be seen against the skyline compared to the deck steelwork viewed against the local countryside; the visual importance of the bridge justifying the additional cost of future repainting of the steel arch. The arch rib is a longitudinally stiffened rectangular steel box that tapers from 3 m x 3 m at the base to 3 m x 1.5 m at the crown. It is fully accessible inside for inspection and maintenance.

The 22 arch hangers were connected to the steel concrete composite deck using 1.5 m deep transverse beams at 7.5 m centres with outriggers projecting 3 m beyond the edge of the bridge deck. Connecting the transverse beams and supporting the deck sections that were not directly supported by the arch were two 1.5 m deep longitudinal steel beams set 10.5 m apart. Secondary transverse beams 0.6 m deep were installed at 3.75 m centres to carry the concrete deck to the main steelwork. Although originally conceived with cantilever outriggers, the lateral forces from the diagonal hangers necessitated adding a trimmer girder that connected the tips of all the outriggers to the longitudinal girders.

The hangers were 84 mm diameter fully locked coil cables with fork ends connecting to lugs at the underside of the arch and spherical socket ends connected to the transverse beams. The fork ends were aligned with the vertical plane of the hanger and the socket ends perpendicular to that plane in order to allow freedom of articulation of the hanger itself and keep the angle at the fork end within the permitted deviation.

During detailed design development conversations with Welsh Government and Public Health Wales led to a reappraisal of the suicide prevention at the bridge. A risk assessment showed that suicide was a notable risk given the prevalence of social deprivation in the area, the iconic status of the structure and the height above the valley. The realisation by the design team that preventing access to a means of suicide was a valuable option that could give an individual more time to seek help, led to adopting bespoke 2.1 m high pedestrian parapets with an anti-climb cap.

2.7 Superstructure construction

Construction was undertaken in stages, working from the north and then the south, as these were the only practical areas available to site the crane required and have space for assembly of the steelwork. Three temporary trestles were required to support the deck steelwork; one located between the existing A465 and the SSSI, one in the Tramroad and one on the new A465 carriageway. Starting with the deck, the first two panels were lifted into position from the north, before derigging the crane and transferring to the south side for the other two panels.

The concrete composite deck was then cast, with precast deck panels used for the parapet cantilevers. It was essential to cast the deck before connecting the hangers as the deck provides in-plane stability to the asymmetric diagonal hanger forces. Permanent restraint preventing the deck from twisting in plan is provided with guide bearings at the abutments. With the deck cast, it was possible to extend the trestles upwards transferring the load through the deck steelwork to the lower trestle sections. The trestle then could provide support to the four steel arch sub-sections. These were lifted into place using a Sarens Gottwald AK680 1200 tonne crane with up to 500 tonne superlift for the largest piece of 132 tonnes installed at 70 m radius.

Figure 2.3 Weathering steel deck erection from the north

courtesy of Costain

The first lift took 6 hours despite the blizzard like conditions. The second lift was delayed 24 hours to allow the storm force winds to abate but then only required a further 5 hours to complete. The third and fourth lifts were easier and completed one per day. Temporary bolted connections between the arch sub-sections were used for speed and to release the crane, as well as to allow adjustment prior to full welding of the permanent connections. Final adjustments were made at the springing by having a temporary support frame around the base of the arch before final grouting of the connection to the foundations.

Figure 2.4 Crane erection of the final arch section supported on temporary trestles

courtesy of Costain

Figure 2.5 Arch base connection with temporary steelwork

courtesy of Costain

Once the deck and arch had been installed, the upper trestles supporting just the arch were removed, and the hangers were installed with enough tension to remove the catenary sagging effects. To achieve the correct final hanger tension the installation process, proposed by the fabricator, helped avoid iterative tensioning operations on the hangers. As it was considered difficult to achieve the correct hanger tension on one hanger without affecting the adjacent hangers, the proposal was that the hangers would be tensioned simultaneously.

To match the hanger forces defined in the analysis model, a precise hanger installation length was specified for each hanger. This differed from the actual physical length by the amount of deflection that would be achieved once the deck permanent load was applied to the arch. The length measurements were critical to achieving the correct tension, so the as built length prior to hanger installation was verified using precise surveying. By creating an effective pre-tensioning length in the cable, the correct cable forces were achieved simultaneously in all hangers as the deck was gradually de-jacked from the lower trestle supports.

To confirm the tensioning had been carried out correctly, site tests were used to show the total force in all hangers on each side of the deck were within 2% of the design values and that the deck profile was correct. Some individual hanger forces did not match, but back analysis of the measured tensions showed only minor differences which were within the design limits. Site measurements of cable deflections also showed that Stockbridge dampers were required on the longer hangers in order to avoid excessive vibration during periods of high wind speed.

Figure 2.6
View along the link road through the arch

courtesy of Roger Donovan/Welsh Government

2.8 Feedback

In recognition of its importance, the bridge was named The Jack Williams Gateway Bridge on 21st January 2019 in honour of Company Sgt Maj John Henry Williams VC, DCM, MM & Bar. He won the Victoria Cross in Oct 1918 at Villers Outreaux, France. Jack, from nearby Nantyglo in Blaenau Gwent, was a colliery blacksmith before enlisting and would probably have appreciated the steel construction that went into the bridge carrying his name.

Transport deputy minister Lee Waters said: “Jack Williams was a true hero and naming such an iconic bridge built in the area he lived after him is a fitting tribute to a man whose name should never be forgotten”.

Figure 2.7
Jack Williams VC
DCM MM & Bar

courtesy of Blaenau Gwent Council

3. UNIVERSITY OF NORTHAMPTON WATERSIDE CAMPUS

3.1 Facts and figures

Client:	The University of Northampton
Design and Build Contractor:	VolkerFitzpatrick
Consulting Engineer:	Tony Gee and Partners LLP
Architect:	Moses Cameron Williams (MCW) Architects
Steelwork Subcontractor:	Briton Fabricators Ltd
Construction Commenced:	2015 - 2016
Public Opening:	2017
Steel Tonnage:	220 tonnes
Protective Treatment:	Type II and Type III (internal parts of box girder) to 1900 Series
Further Information:	https://www.tonygee.com

Figure 3.1 The University of Northampton Waterside Campus road bridge

courtesy of Tony Gee and Partners

3.2 Design basis

Design Standards:	BS EN 1990 to 1994 and 1997 with corresponding UK National Annexes Design Manual Road and Bridges Manual of Contract Documents for Highway Works
Loading:	Load traffic actions and other actions specifically for road bridges, as set out in BS EN 1991-2:2003 and the UK National Annex. Specifically, Load Model 1, Load Model 2, Load Model 3 (SV80) and Load Model 4
Design Life:	120 years

3.3 Location

As part of a new University Campus development in Northampton, a new road access bridge was required to span over the River Nene.

The client's aspirations and the planning requirements were set to keep the character of the existing landscape while creating an appropriate landmark structure for the new campus.

Figure 3.2
Location plan

courtesy of Tony Gee and Partners

Figure 3.3
Aerial view during construction

courtesy of Commission Air

3.4 Bridge context

Right through from the planning phase, it was considered that the bridge would need to respect the character of the existing landscape and the river, which was defined as “one of elegance”. The simple, quiet and understated setting shouldn’t be compromised by an over-stated bridge design, over engineered or “iconic” for the sake of being “iconic”. The conceptual design development process researched several different forms that would fit this remit. Ultimately it was considered that the bridge design must be inspired by an appreciation of its setting, the appropriateness of the scale of statement and an appreciation of other bridges that span the River Nene. A desktop review of some of the existing river crossings showed a typology that supports this thesis – often simple arch structures, in stone, concrete or steel/ironwork, the existing crossings are responsive to the local river conditions, with a subtlety to their detailing and innovation in their structural design. The selected form for the new crossing was an elegant shallow arch structure that would also be sympathetic to its surroundings.

Formed from two arch spines, the structure has been designed to meet the skew alignment of the road respective to the river, thereby minimising the extent of modification required to the banks. The twin arch structure was developed to avoid the “tunnel effect” of many pedestrian/cycle underpasses. The form enables short and mid distant views of the river, embankments, and campus buildings to create an airy pleasant walkway. Minimum headroom and air-draft clearances were specified for the main span and walkways with upper limits on section depth imposed by the road alignment. Three-dimensional modelling was undertaken to ensure the most efficient use of the room available to retain an elegant and efficient structure.

The bridge design had also to address several challenges as listed below:

- Accommodate a road alignment that would tie-in with the Bedford Road Junction and the new campus.
- Span without any support in the River Channel (48.5 m min) and provide an 18m wide navigation channel with 3 m clearance above normal water level.
- Meet the flood risk design criterion for a 1 in 200 years return period.
- Minimize disruption to the extensive number of buried services (11 kV cables across the river, 33 kV and 132 kV in the North bank).
- Maintain a river navigation during the construction period.
- Minimize disruption to the river to maintain the ecology and biodiversity.
- Create a safe and pleasant pedestrian/cycle environment along the riverbanks.
- Address cost, statutory authority, and buildability issues.

Arch bridges are normally aesthetically attractive structures. The arrangement adopted for the Northampton University Bridge could easily be compared to the engineering wonders of Robert Maillart's elegant deck-stiffened arches. The span and configuration of the Nene Bridge seem to be comparable to the Swiss Engineer's creations such as the Tavanasa Bridge or the Vessy Bridge. In the UK, it could also be compared to the M25 Runnymede bridge. A comparison of arch slenderness between these selected structures is included in the table below, which clearly demonstrates the efficiency of the steel-concrete composite deck adopted for Northampton University Bridge.

Table 3.1 Arch bridges – Comparison of similar spans (road bridges)

Bridge	Tavanasa	Vessy	New Runnymede	Northampton
Year constructed	1906	1936	1979	2016
Span (arch) in m	51.0	56.0	54.6	50.3
Rise in m	5.57	4.77	6.96	3.61
Span/Rise	9.2	11.7	7.8	13.9
Structural depth in m	0.83	0.83	1.80	1.19

The steel-concrete composite solution improved buildability by reducing the interference with the river and minimising the temporary works requirements. Site constraints included the shallow river needing to remain open as much as possible, and crane access being limited to the riverbanks due to the presence of existing high voltage cables running across the riverbed.

The bridge was set skewed between the riverbanks, minimising the extent of modifications to the river and therefore helping to maintain the ecology and biodiversity. The structure incorporates a safe and pleasant pedestrian and cycle environment that has also been created along the riverbanks. The solution allows safe passage of vessels with an 18 m wide navigation channel and 3 m headroom above normal water level. During construction, a temporary 11 m wide river navigation was maintained. The bridge geometry also includes a clearance envelope beyond the navigation requirements to meet the flood risk design criterion for a 1 in 200 years return period event. The steel design refinement permitted use of smaller and more readily available construction machinery and significantly reduced the requirement for temporary works. An estimated embodied CO2 equivalent indicates that the steel composite solution has a lower total impact than the originally proposed concrete design.

3.5 Bridge conceptual design

During the tender design competition, close reviews of the flat arch concrete solution were undertaken by the design and build team, concluding that the imposed loads to the foundations and the construction method required to erect a concrete structure would not prove to be economical. The significant vertical and horizontal imposed loads to the foundations would be caused by a combination of the very high span to rise ratio and a heavy deck. In addition, the deck slenderness requirement and tensile force induced into the approach spans would have required the use of post-tensioning concrete in the deck, which necessitates fully grouted tendons with provisions of means for access during future inspection of prestressing anchors. Finally, as the river needed to remain open crane access from the riverbanks would be limited making installation of heavy precast concrete units challenging.

An alternative design was developed using a steel-concrete composite structure that would meet planning and employer's requirements but with a lighter structure to reduce impact on the foundations. The aim was to also improve the structural stiffness required for the flat arch as well as facilitate a simpler method of construction to reduce the costs of temporary works.

The awarded solution includes 220 tonnes of welded steel plates to form a shallow and flat arch structure with deck cross girders. The connection between superstructure and substructure is integral. Due to the shallow clearance above the water, and for aesthetic reasons, weathering steel could not be adopted thus the steel was specified with a protective coating. Openings have been provided along the webs of the box girder to provide access for inspection of the internal parts of the steelwork.

The lighter deck offered a significant reduction in foundation loads, and the erection of steel girder sections allowed simplification of the erection process. Cranes could be used on both banks, avoiding the need for extensive river works or a temporary pontoon.

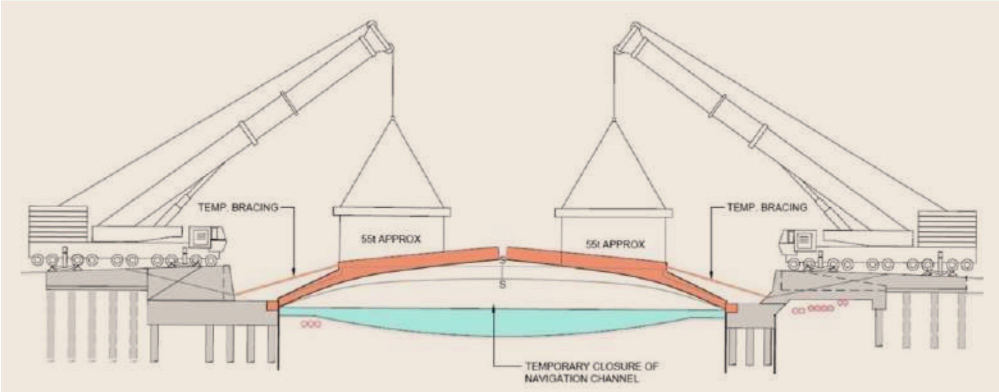


Figure 3.4
Main girders installation method

courtesy of Tony Gee and Partners

The use of steel girders also eliminated the need for post-tensioning as the axial tensile force in the approach spans is resisted by the main girders with passive anchorage into the abutments.

In addition, the overall arrangement was optimised by bringing the abutments forward, which shortened the approach spans and therefore reduced the overall length of the superstructure by approximately 30%. This arrangement also permitted use of the earth backfill material as ballast, preventing any net axial tension in the piles. The change of the structural arrangement and the form of construction reduced the foundation loads due to the self-weight of the structure by approximately 40%. With lighter steel girder sections to erect, the solution also significantly simplified the installation of the primary spanning structure while the in-situ concrete slab could be cast using conventional methods. This weight reduction not only offered ease of construction but also contributed to the reduction of material required.

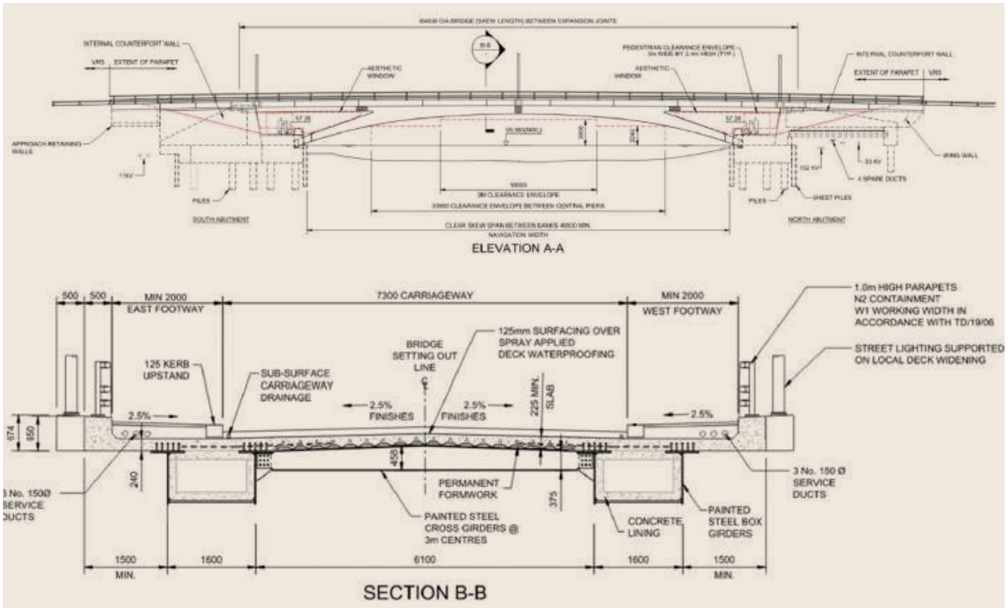


Figure 3.5
Bridge elevation and typical cross-section

courtesy of Tony Gee and Partners

The setting-out of the foundations had to respect the presence of a considerable number of underground high voltage electricity cables associated with the adjacent site of an old power station. The new substructure configuration negated the need for any utility diversion, mitigating associated risk to the construction programme.

3.6 Design details

The 12.3 m wide steel-concrete composite deck includes a ladder deck with two rectangular open top box girders with 1500 mm edge cantilever slabs. The arch box girders have a constant width of 1600 mm and a variable depth from the springing points to midspan. At the support points the steel box is approximately 700 mm deep and increases to an overall maximum of 2120 mm where the arch fuses with the deck to finally reduce to 965 mm at midspan. At the arch-deck interface, the intrados flange and the webs were made fully continuous, while the top flange of the box arch section gradually disappears via internal horizontal stiffeners lined-up with the access openings on the web as the arch meets the bottom flange of the tie beam.

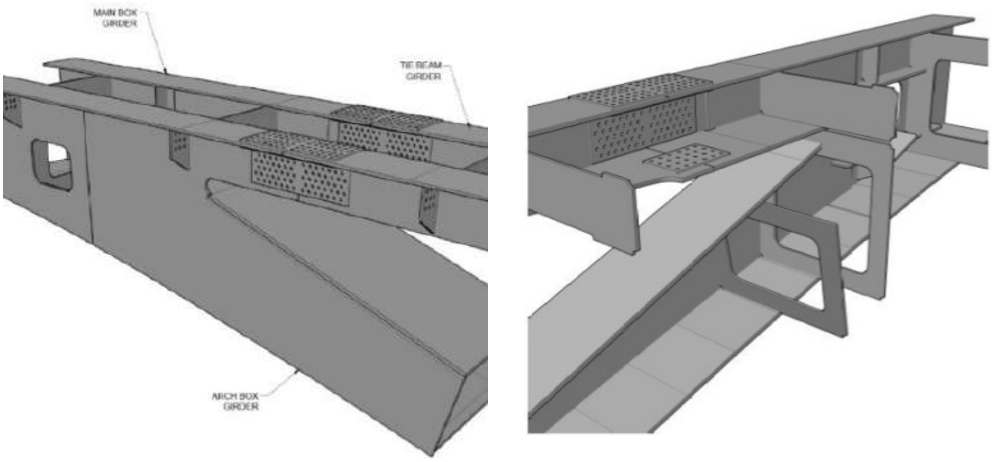


Figure 3.6
Box girder details at arch-deck connection

courtesy of Tony Gee and Partners

As the internal clear span between the girders would have been excessive for a reinforced concrete slab, steel cross girders were introduced to act compositely with a slab cast on top of permanent formwork. Simple connections could be adopted with the cross beams set perpendicular to the main girders, as well as a neat interface between the permanent formwork and the steel flanges. This layout also allowed the deck reinforcement to be fixed orthogonally to the cross girders without difficulty.

The arch articulation is a pseudo two-hinged system. Fully integral continuity of the steel boxes with the reinforced concrete abutment at the arch springing would have required the use of embedded pre-tensioned bolts to provide an adequate moment connection. Tight tolerances in the setting-out of these bolts within the concrete would also have been required to align with the erected steelwork, with careful allowances made for geometric deviations of the various components. Therefore, a simpler connection detail between arch springing lines and substructure was adopted to improve constructability. The introduction of hinges at the springing lines also eliminated redundant moments due to restrained deformation.

The general approach proposed by the design team was for a pocket to be formed into which a springing shoe could be inserted, grouted up around packs on the bearing surfaces, and otherwise encased in concrete (see Figure 3.7). This detail would provide some end embedment, and a clean visual and maintenance free finish. A range of shoe and pocket details was discussed with the contractor VolkerFitzpatrick, and the preferred solution was a steel plate lined pocket system to be incorporated into the pile cap. This was considered to best fulfil several requirements, namely the possibility of accurate positioning of the pocket, providing a robust surface from which to potentially jack during installation, and an opportunity to spread the loading and reduce local stresses in the concrete.

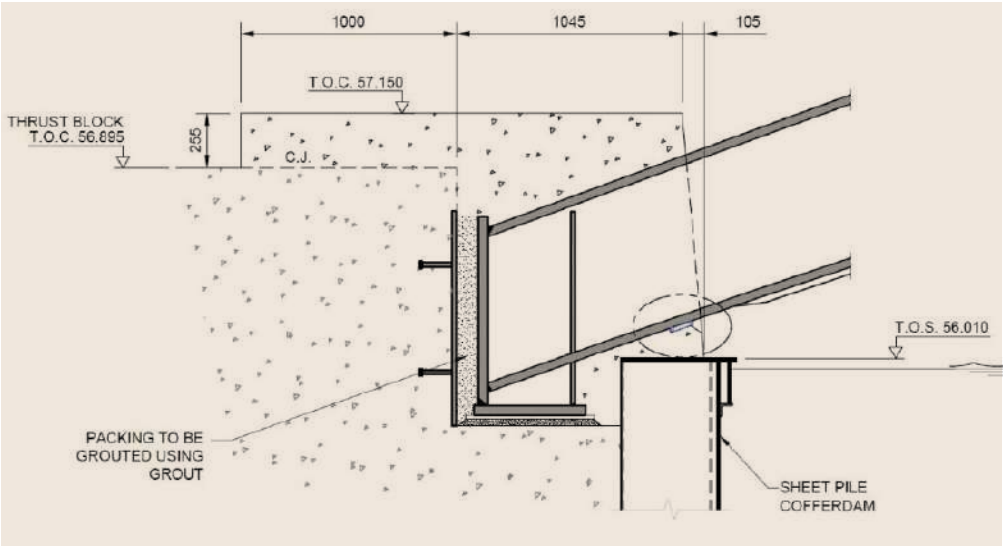


Figure 3.7
Arch springing detail

courtesy of Tony Gee and Partners

The integral connection between the tie beams and the abutment was achieved by extending the steel girders into the substructure. Tension from the steel girders was transferred into the reinforced concrete sections using a combination of shear studs and direct contact pressure between the steel diaphragms and surrounding concrete.

Although using weathering steel to avoid the need for a coating was considered, the 'rusty' appearance was deemed not acceptable to meet planning requirements, and the shallow clearance above the river would have required a Departure from Standard[1]. Therefore, a traditional coated steelwork solution was adopted.

Facilitation of access for future maintenance was also considered. The inclined and lower parts of the arch include a welded rectangular steel section in-filled with self-compacting concrete. Inside the deepest part of the box girder is accessible via discrete openings in the webs located on the inside faces. The braced pairs of girders within the side spans can be directly accessed from the riverbank. The central portion of the span is too shallow for internal access; therefore, a concrete lining was provided to the inside faces of the girder. The concrete lining solution and the infill also provides additional robustness against impact from a 50 tonne barge.

3.7 Superstructure construction

As part of the structural steelwork fabrication process a trial erection was undertaken in Briton’s yard, to confirm the ability to assemble all the main elements.

Figure 3.8
Trial erection

courtesy of VolkerFitzpatrick

The springing detail needed to take account of the installation methodology and be able to absorb the construction deviations to ensure the arch levels at mid span could be accurately set. While clearances to the navigation channel air draft envelopes were tight, it was also noted that from an aesthetic regard, the flat arch and maximising section depths for the steel work had led to a geometry that avails the maximum possible longitudinal camber.

Figure 3.9
Installation of the arch springing detail

courtesy of VolkerFitzpatrick

A midspan connection was required as well as connections between the main arch girders and approach spans. Tension controlled bolts with dome heads provided a visually discrete solution for these connections. Temporary bracings were required to prevent in plane and out of plane buckling of the bare steel structure during construction.

The main girders were delivered to site in 37 tonne main sections (25 m long and 2.2 m wide) during early August 2016. Thanks to a detail design refinement, the structural steel tonnage was reduced such that each section could be lifted into position using one 500 tonne capacity mobile crane set behind each abutment.

Figure 3.10
Lifting of the two half arches

courtesy of VolkerFitzpatrick

The lifting and lowering of each half arch into position allowed the central bolted splices to be connected using tension control bolts without the need for mid-river propping. The installation of both arches with temporary bracings was completed during a single day.

Figure 3.11
Execution of central bolted splice connection

courtesy of MCW Architects

With the main structural steel completed, the concreting phase could start with the arch springing and the integral connection between the tie beams and the abutments. This allowed the lower part of the steel box arches to be in filled using self-compacting concrete cast in a symmetrical sequence, followed by casting of the deck slab on top of the permanent formwork.

3.8 Feedback

The client required a simple yet elegant structure that blended into the existing landscape and the river. With a span to rise ratio of 13.8, the flat arch bridge for the Waterside Campus at the University of Northampton has provided a landmark structure while keeping the character of the existing landscape.

The design and build alternative design offered a steel-concrete composite deck that reduced the imposed loads to the foundations with an improved buildability and economy, working around a considerable number of underground high voltage electricity cables. This elegant solution retained the client's aspirations for the appearance and integration of the structure and was delivered to programme and within budget.

YANDHAI NEPEAN CROSSING

4.1 Facts and figures

Client:	Roads and Maritime Services
Consulting Engineer:	BG&E
Architect:	KI Studio
Main Contractor:	Seymour Whyte Constructions Pty Ltd
Steelwork Subcontractor:	Civmec
Construction Commenced:	May 2016
Public Opening:	October 2018
Steel Tonnage:	700 tonnes
Protective Treatment:	Epoxy Polyurethane Multi-Coat System
Further Information:	https://www.bgeeng.com

4.2 Design basis

Design Standards:	Australian Standard AS 5100 Bridge Design Sétra[3] and Hivoss[4] dynamic design guides
Loading:	AS 5100-2 Pedestrian & Cyclist Live Loading AS 5100-2 Flood Loading Bespoke Wind Tunnel Testing
Design Life:	100 years

4.3 Location

The new bridge forms the Yandhai Nepean Crossing over the Nepean river at Penrith, New South Wales located at 33°44'49.30"S, 150°40'52.54"E. The bridge spans in an East to West direction over the Nepean River between Penrith, located to the East and the Emu Plains located to the West, providing a link between River Road and Memorial Avenue.

The bridge is situated between the Victoria Road Bridge carrying the A44 Great Western highway 200 m North, and Regentville Road Bridge carrying the M4 located 2.8 km South.

Figure 4.1 Location plan of Yandhai Nepean Crossing

courtesy of Seymour Whyte Constructions

Previously the community used the existing Victoria Road Bridge as a crossing point, however Transport for New South Wales (NSW) identified that the state of the crossing at this location was inadequate to meet current design and safety guidelines. Demand for a safer crossing point for foot and cyclist traffic over the Nepean River in Penrith grew following extensive community lobbying.

The new bridge forms a critical connection as part of the New South Wales Bike Plan, and provides a dedicated safe river crossing, improving access for pedestrians and cyclists between the Emu plains and Penrith in one of the fastest growing parts of Western Sydney.

4.4 Bridge context

Prior to the construction of the Yandhai Nepean Crossing, the only viable crossing point for Residents of Penrith was the Victoria Road bridge, which was constructed in 1867. However, this crossing is not suitable for high volumes of combined pedestrian and cyclist traffic, being unable to meet modern safety standards due to the proximity to vehicular traffic. The lack of a safe crossing point for cyclists and pedestrians presented a major constraint in delivering a comprehensive bicycle network for Penrith.

Consultation began with Penrith City Council and local stakeholders to provide a new separate crossing point. The concept design was developed as part of a Road and

Maritime Services commission for a potential structure, with the final design awarded to the twin reverse truss concept. The invitations for detailed design were issued in 2014, with BG&E being awarded the detailed design in 2015, and construction commencing in May 2016.

The $50 million (AUD) project involved constructing 455 m of shared path as part of a 7 km loop section of the Great River Walk. The bridge forms approximately 260 m of the shared path, including a 200 m single main span across the Nepean River.

From concept stage it was clear that structural steel was the ideal choice for the construction, as it facilitated the 200 m long span and enabled the bridge to satisfy its iconic design and architectural requirements as a light and elegant structure, whilst providing a practical and economical engineering solution.

The bridge was constructed using a triangular warren truss profile for the main span with curved steel & concrete composite approach spans. The fabricated steel truss boasts artistic lighting, architecturally designed hand railing, and vantage points to provide a visual focal point within the region.

Figure 4.2 Yandhai Nepean Crossing end view

courtesy of RMS Photography

The design needed to consider the effects of wind excitation, footfall vibration and flooding. The bridge is subject to some of the most severe flood loads in NSW with stream velocities in the order of 7 m/s in the 2000 Annual Recurrence Interval (ARI) event, with further increases anticipated. This proved challenging when designing for flood forces on a structure of this size and called for sophisticated structural analysis to ensure a robust design.

The bridge was successfully completed in October 2018 and opened by the NSW Premier, Gladis Berijiklian. The bridge was named 'Yandhai' which symbolises the walking of the path between past and present, reflecting the traditional stories used by the local Darug people in their portrayal of the Nepean River and its surrounds. The iconic design of the crossing embraced a similar journey, shifting and melding over time from its inaugural concept, embracing a massive steel warren truss with dual-curved geometry, to the ultimate straightened truss, boldly owning its position adjacent to the heritage Victoria Bridge upstream.

4.5 Bridge conceptual design

The Yandhai Nepean Crossing, Penrith Australia, was architecturally designed by KI Studio, to be an iconic structure, providing a critical connection between Emu Plains and Penrith as part of the New South Wales Bike Plan. It is the longest single clear truss span footbridge in the Southern Hemisphere, and the first node by node launched bridge in Australia.

The initial bridge design was strongly driven from an urban design point of view and resulted in a twin reverse curved truss spanning 175 m. The aim of the structure was to create a destination in its own right, rather than just a crossing point, promoting Penrith as a river city.

The design of the truss is arguably one of the most material effective engineering solutions, and while truss bridges are certainly not unique, through the benefits of long span geometry the Yandhai Nepean Crossing was able to utilise a single top chord and still maintain peripheral headroom clearance for the deck, contributing to a substantial reduction in material.

Figure 4.3
Early concept design

courtesy of KI Studio

The primary engineering challenge from day one was constructability. The design needed not only to consider the effects of wind excitation, footfall vibration and flooding, but also how to construct a single span structure over a 200 m wide, busy recreational waterway was key to the project success.

Permanent mid-river piers were undesirable, and the busyness of the waterway proved to be a key constraint to the project. Permanent piers within the river were avoided during the concept design as they would have unnecessary, long lasting negative impacts on the river. The single span used temporary piers, which allowed the Nepean river to remain open with minimal restrictions throughout the entire construction period.

By 2014, at the time of the client inviting tenders for the detailed design, a construction methodology had not yet been established. BG&E's winning bid proposed to incrementally launch the concept dual-curved truss using a node-by-node approach, an innovative method never undertaken before. Node-by-node launching involves using movable supports on tracks/skates rather than conventional static supports, allowing a structure to be launched utilising only four supports at any one time. This method also has the advantage of being suitable for launching any plan arrangement, such as the initial reverse curve plan concept.

Figure 4.4 Constructed Yandhai Nepean Crossing

courtesy of RMS Photography

4.6 Design details

BG&E was awarded the detailed design contract of the Yandhai Nepean Crossing by RMS in 2015. The initial concept design form was initially retained, and BG&E determined the launching corridor, shown in Figure 4.5, to account for the double curved geometry and the swept path during launching. Two sets of tracks were used in this approach to allow movement in transverse and longitudinal directions during launching.

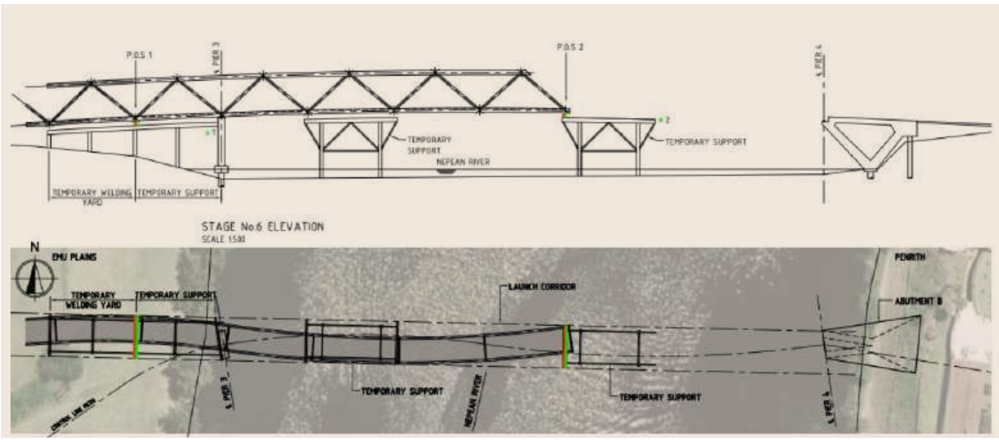


Figure 4.5 Initial curved launching approach

courtesy of BG&E

Following a review of the project's budget constraints the curved design was deemed to be too costly to build. BG&E undertook a comprehensive framework of value engineering to reduce the estimated cost of the developed concept design to within budget constraints. BG&E presented several cost saving options, including straightening the truss from the concept alignment of two reverse curves, and lengthening the span from 7 to 8 bays, increasing the span from 175 m to 200 m. The longer span was achieved whilst reducing the overall tonnage of structural steel by utilising strategic localised strengthening, improving the efficiency with which materials were used and thereby reducing the cost and embodied carbon requirement for the construction.

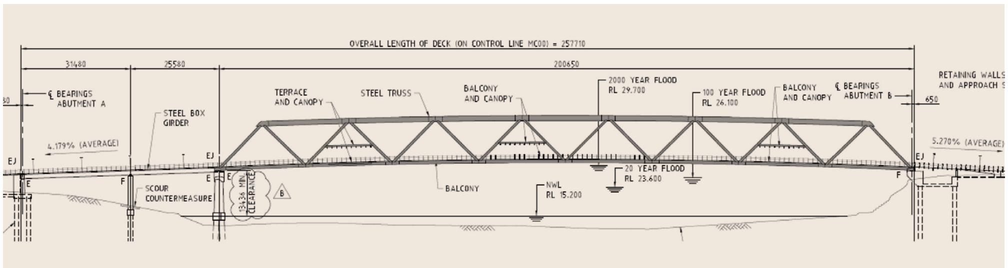
During design development, the complex Eastern abutment structure was removed and replaced by a simpler concrete box structure higher up the slope of the riverbank. Removal of the complex abutment also improved the stability of the steep riverbank. The simpler concrete box abutment also doubled as the launching structure, allowing the long steel truss to be launched as a series of connected modules and removing the need for a separate temporary launching structure making the approach more cost effective.

The 200 m main span was constructed using an S350 steel warren truss comprising 8No. 25 m bays. The truss is triangular in cross section utilising a single top chord and two bottom chords with a 14 m construction depth throughout. The main structural elements are fabricated from circular hollow sections ranging from 750 mm to 1450 mm in diameter, rolled and welded using the submerged arc welding process. The bottom chords support circular hollow sections forming the plan bracing, and square hollow sections forming the cross-girder mountings for the deck structure.

The approach spans located at the Emu Plains end comprise a continuous steel box and concrete composite construction, with the Penrith approach comprising a reinforced concrete abutment / bank seat.

The bridge deck provides a continuous 4.6 m wide shared pathway. Observation balconies are located along the main span where the deck has been locally widened and overhead canopies provided at these locations. The parapets comprise architectural designed handrails which are continuous along the full length of the bridge.

Figure 4.6
Final design general arrangement elevation



courtesy of BG&E

The protective coating comprises a 5-layer epoxy-polyurethane system. The system included a primer layer, followed by 3No. epoxy layers and a polyurethane topcoat. This system enabled the intended architectural colour to be achieved whilst being protected from long term UV exposure.

The point-by-point construction process required eccentric support conditions and temporary jacking requirements. To prevent crushing under concentrated loading and to ensure efficient use of material during operation, internal plate stiffeners were provided locally to the bottom chords and end bay diagonals.

At the bottom chord nodes, the circular hollow sections were internally stiffened using full diameter diaphragm plates. Stiffeners were avoided at the top chord nodes to simplify construction, with the preferred solution to locally increase the wall thickness of the chord. Diaphragm plates were also provided at the plan bracing and cross girder connections, to stiffen these joints and reduce weld stresses when subject to transverse forces.

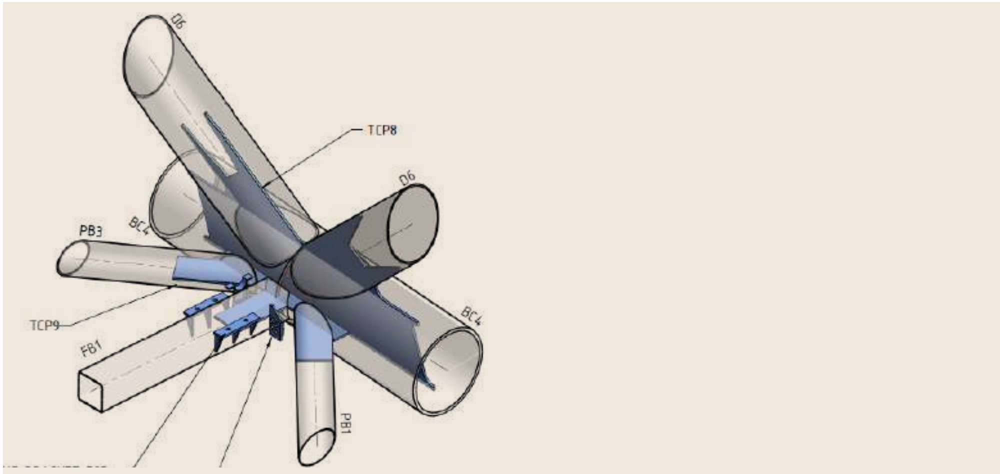


Figure 4.7 Bottom chord node detail

courtesy of BG&E

To cope with the high axial forces, non-standard elements were required to achieve the design resistance requirements and the BG&E design team sought approval from the Client to use an alternative Australian Standard for the calculation of steel section capacities. The AS 5100 bridge code uses a simplified method of determining steel capacity, typically more suited to steel/concrete composite decks, and therefore can be conservative when considering section slenderness. An approach based on Finite Element Analysis to determine element and joint stresses resulted in a reduction of the steel tonnage to 700 tonnes. It is estimated that the original structure with conventional design and construction techniques would have required in the order of 1150 tonnes.

A comprehensive dynamic analysis of the truss identified a potential issue with footfall vibration. The analysis considered loading due to crowds, applied with the aim of exciting particular natural frequencies of the structure within typical footfall range. The forcing frequencies and load distributions were adjusted to get the most adverse response for the mode of vibration being considered. The resulting structure response was compared to comfort limits obtained from SÉTRA[3] and Hivoss[4] design guides.

Based on the results, it was found that there was a risk that levels of vibration in both vertical and horizontal directions could exceed comfort limits for several modes, particularly when exposed to combined footfall and wind excitation as wind buffeting off the top chord was exciting torsional modes.

Rather than providing additional material to stiffen the structure, which would have had significant cost implications, tuned mass dampers (TMDs) were designed to limit vibration amplitudes as part of a more cost-effective solution. Based on results from the dynamic analysis including TMDs, vibration amplitudes were found to decrease by approximately 90% to then fall within the acceptable range.

Once the bridge was constructed, the mass damper systems required tuning to ensure effectiveness. BG&E pursued a real-world testing approach by producing horizontal excitation within the structure by tethering it to the Victoria Road bridge upstream. This tethering approach allowed the bridge to be laterally loaded with the bridge experiencing 20-30 mm displacement followed by sudden release. This allowed the frequency response of the structure to be observed in a real-world scenario and the TMD’s to be optimised. This solution allowed BG&E to bridge the gap between theory and practice to develop an efficient, resilient and safe structure through innovative design.

4.7 Superstructure construction

BG&E’s winning bid proposed to incrementally launch the truss from the Eastern bank using a method never seen before in Australia, node by node launching.

Conventional incrementally launching (pushing a structure over stationary supports) of trusses is seldom undertaken due to the impact on the bottom chord. This is due to large point loads from the launching supports imposing significant bending moments and localised stresses along the bottom chord during construction. Accommodating these results in additional steel tonnage that is only required during the launching phase, to prevent localised buckling (particularly in circular hollow sections), creating consequentially uneconomical designs.

Node-by-node launching utilises movable jacking support cradles mounted on tracks (or skates on rails) that pass each node of the truss forward while only ever supporting the truss in four locations. This technique allows any plan alignment to be launched as the movable supports only move in a straight line back and forth passing the nodes forward, this differs greatly from conventional launching where constant curves must be launched when using stationary supports.

Using only four support locations has the advantage of allowing the structure to maintain static determinacy throughout the temporary launching stages, simplifying the reactions on the temporary works and reducing the need for settlement control and deflection monitoring.

This solution was described by the client as ‘a masterstroke by BG&E by providing an innovative and smart construction methodology, which put the project on the road to success’.

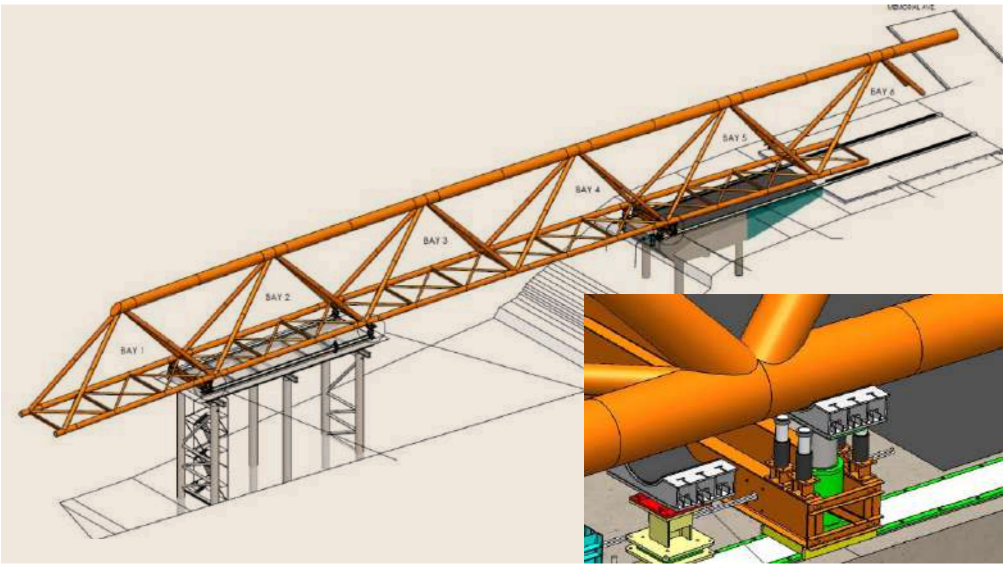


Figure 4.8
Revit model view showing detail of support cradles

courtesy of BG&E

Considering the inherent cantilever strength within a truss and allowing the truss to act like a launch nose and cantilever between temporary supports, a cost effective and sustainable solution was born.

The main span was arranged into 8 bays of 25 m. Structural analysis was undertaken to determine the maximum cantilever that could be achieved before additional strengthening would be required and resulted in a possible cantilever of 50 m.

Geometrically this required two temporary pier tables in the river to facilitate the node by node launching of the 200 m span, utilising the 50 m cantilever three times and being passed over 2 x 25 m long pier tables. Each table needed to be approximately 1 bay long (25 m) to allow the nodes to be passed forward. Stationary cradle supports were fixed to one end of the table and supported the truss while the movable beams retract backwards to pick up the next node 25 m away, hence the tables needed to extend at least 1 bay long, with intermediate spacing not exceeding 2 bays in length.

The temporary piers comprised 3No. braced steel trestles, each comprising 1.2 m deep double web steel welded beams atop braced steel piles designed to withstand 1 in 100-year flood events, potential river vessel impacts, and the 150 tonne concentrated loads from the node by node launching. The trestles were required to be sufficiently stiff to restrict movement to just 3 mm during loading phases.

The geology of the Nepean riverbed presented a significant challenge to piled construction as the pile casings had to be cored directly into rock whilst maintaining structural tolerances.

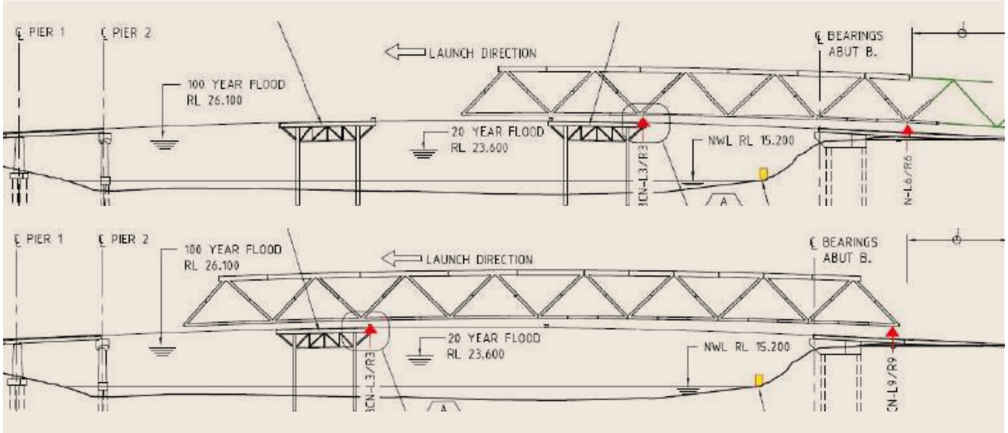


Figure 4.9 Launching Construction Stages showing temporary pier tables and launching supports

courtesy of BG&E

For the main truss construction, the bottom chords, cross girders and plan bracing were prefabricated off site in single 25 m by 8.2 m bay sections, with the diagonals and top chord fabricated separately. These sections were transported to the on-site construction facility built surrounding the Eastern bank seat, where they were assembled using temporary welding, and surveyed to ensure the correct profile before being submerged arc welded, painted and launched.

An enclosed workshop was constructed over the Eastern bank seat with 2No. 100 tonne overhead gantry cranes used to provide safe and reliable lifting for the construction. The workshop also featured a hinged temporary works structure to enable efficient construction by allowing the steel segments to be held in place prior to temporary welding, final surveying and full submerged arc welding of the sections. This approach reduced the construction programme by two months.

The truss was constructed during the summer of 2018 and although thermal expansion of the steel truss was anticipated, a severe heatwave during construction presented unforeseen challenges due to differential thermal expansion. This was particularly challenging for the surveying team as the truss would expand up to 150 mm during the day with exposed truss element experiencing significantly more expansion than those inside the workshop. In order to overcome this, the surveyed reference points had to be limited to those within the workshop out of direct sunlight, and in the very early hours of the day before the sunlight could heat the external elements which would be conducted along the constructed span and factor measurements based on the steel's thermal expansion coefficient.

4.8 Feedback

In 2018 at the bridge opening, Minister Ayres thanked the project team 'who have set a new standard in construction and engineering excellence.'

In a testimonial to BG&E, Ian Allan the Project Director at Roads and Maritime Services stated, 'This innovation was critical in bringing the project within budget while also achieving the required levels of safety during construction that would not have been achieved without an incremental launched solution. Before BG&E's node-by-node technique, there was no clear way to construct the bridge within the allocated project budget, or within the required safety and quality criteria. It is highly likely that without the innovation, the project would not have secured the funding to proceed to construction.'

In 2019, the Yandhai Nepean Crossing received a prestigious good design award winner accolade in the architectural design category in recognition of outstanding design and innovation. It was one of almost 700 nominated design projects evaluated internationally.

In 2019, the structure was highly commended by the Consult Australia Awards for Excellence in Design Innovation.

In 2020, the Yandhai Nepean Crossing was listed as a finalist by the Sydney Chapter of the Australian Engineering Excellence Awards (AEEA).

GARRISON CROSSING

5.1 Facts and figures

Client:	CreateTO on behalf of City of Toronto
Consulting Engineer	Pedelta Canada
Architect:	DTAH
Main Contractor:	Dufferin Construction
Steel Tonnage:	~350 tonnes of duplex stainless steel (grade 1.4462)
Steel Manufacturer:	Industeel (subsidiary of ArcelorMittal)
Steelwork Fabricator:	Mariani Metal Fabricators
Fabrication Commenced:	August 2016
Public Opening:	October 2019
Further Information:	https://www.pedelta.com

5.2 Design basis

Design Standards:	CAN/CSA S6 Canadian Highway Bridge Design Code AISC Design Guide 27[5] EN 1993-1-4:2006+A1:2015
Loading:	CAN/CSA S6
Design Life:	75 years

The crossing comprises two bridges which incorporate a unique arch design consisting of a tied stainless steel network arch with a distinctive crossing diagonal hanger pattern, and a triangular profile, with a single arch rib inclined at 18° to provide a slender, transparent and elegant structure.

Durability was a significant issue to consider for this project, as the bridges are permanently exposed to a potentially corrosive environment due to the application of de-icing salts in winter. The maintenance for stainless steel structures is limited to regular pressure washing with water to clean the structure from de-icing salt accumulation. The duplex stainless steel grade used ensures high corrosion resistance. In addition, the use of stainless steel is particularly beneficial for structures with significant maintenance constraints, such as bridges over a railway. It eliminates the need for major maintenance and the associated costs, including indirect cost arising from disruption to the users during repair proceedings.

Figure 5.1 The Garrison crossing

courtesy of Pedelta Canada Inc

The design was focused on both structural efficiency and proportioning the geometry in such a way that is aesthetically pleasing to the eyes of the public, resulting in a span-to-rise ratio of around 6 and a span-to-arch depth ratio of around 100. Both bridges use trapezoidal cross sections for the girders that make up the deck, and triangular cross sections for the arch ribs.

5.3 Location

The bridge is located just west of the main downtown area of Toronto, and provides a link from the city's Stanley Park to the historic site of Fort York and the waterfront, crossing above two of Canada's busiest rail corridors, and linking parks and green spaces.

Figure 5.2 Plan view showing location of bridges

Located just west of the main downtown area of Toronto, Canada, the 17.4-hectare Fort York historic area is an oasis of calm in a vibrant city. It dates from 1793, when the small town was known as York, and served as a defence for the harbour being manned by a garrison of soldiers. The site today is a major tourist attraction, and the green space known as Garrison Commons is very popular with city residents. However, access to the site for pedestrians and cyclists was by a busy road bridge, which crossed two major railway lines.

5.4 Bridge context

A Design-Build procurement model to facilitate optimal and cost-effective construction was chosen for this project. The design development started in the spring of 2016 and was completed in the autumn of 2019.

The bridges were designed to add a distinctive visual element with a clear identity to the city of Toronto, without dominating the skyline and natural beauty of the surrounding neighbourhoods and parks.

One of the key challenges was designing and building the bridge over the existing railway corridors, placing the substructure outside the right-of-way of the corridors and keeping a vertical clearance of 7.44 m above the top of the rails. The bridges needed to have an unobstructed width of 5 m to accommodate pedestrians and cyclists. They cross over two active rail corridors, so consideration was given to the protection, safety, and security of both the railway operations as well as the pedestrians and cyclists using the bridge.

The south approach is situated on the Garrison Common-Fort York Area. Garrison Common is a wooded open space west of the walled Fort that is historically significant as one of the important battlefields in the War of 1812 (also known as the Battle of York). To reduce the impact on the landscape of Fort York, this heritage site of significant cultural importance, the bridge and approach ramp within Garrison Common at Fort York were designed and built with the minimum footprint that could possibly be achieved.

5.4.1 Why stainless steel?

There is an initial cost premium for stainless steel when compared to traditional carbon steel. However, unlike galvanized or painted carbon steel, the naturally occurring corrosion-resistant chromium-rich oxide film means there is no requirement for applying a protective coating. Eliminating the need for coating maintenance or component replacement due to corrosion can lead to significant long-term maintenance cost savings.

The lifecycle cost was one of the key points considered at the preliminary design phase, when the use of a stainless steel option was evaluated from an investment perspective. The design was driven by utilizing less material, providing an extended life span and easy maintenance even if the initial cost was slightly higher. The use of stainless steel increased the initial capital cost by approximately 12% compared to the same design concept in carbon steel. The lifecycle cost analysis at the tender phase, however, concluded that the initial higher construction cost of stainless steel was balanced by the extended lifecycle of the more corrosion resistant bridges, and the lower maintenance would reduce the overall cost of ownership. This represented a net advantage for the owner, in addition to improving safety and long-term durability.

The total embodied CO2 in stainless steel associated with the whole cycle from raw acquisition and production depends heavily on the steel production method with Electric Arc Furnace steelmaking giving lower embodied carbon. However, to get a true picture a comparative life cycle assessment is needed to assess the CO2 emissions and energy consumption considering the specific project constraints and other considerations such as the higher strength and extended lifespan of duplex stainless steel than carbon steel, or the environmental impact of coatings[6] .

5.5 Bridge conceptual design

The engineering considerations included the following; functional requirements, topography, geotechnics, limitations of space, alignments and clearances over the railway, impact on the adjacent existing elements and utilities, design standards, vibration limits, wind demands, snow removal, durability requirements, sustainability aspects, and constructability challenges.

Each bridge is supported by a single arch rib inclining at 18° to provide a slender, transparent, and elegant impression. The two arches tilt in opposite directions which results in a dynamic expression. The stainless steel provides premium aesthetics and a safe and durable asset for the City of Toronto.

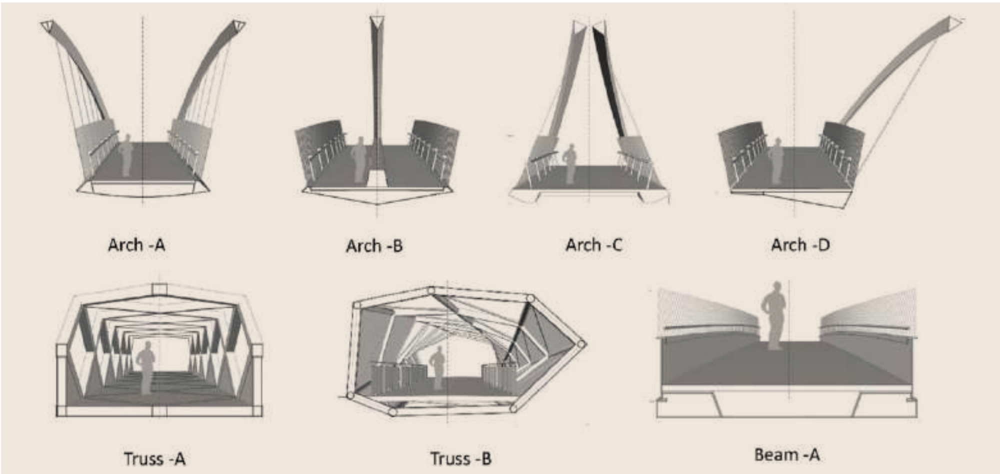


Figure 5.3 Structural concepts/ forms considered

courtesy of Pedelta Canada Inc

Various bridge layouts were explored to identify the most appropriate location of the two railway crossings and connecting paths. The process continued by exploring a series of suitable bridge types and materials, with particular attention to aesthetics. Some of the concepts considered can be seen in Figure 5.3. Three-dimensional modelling of the bridge and surroundings was used to visualize the alternatives. For each concept, sketches, drawings, and preliminary structural calculations were prepared in order to estimate material quantities and confirm that the concept could later be developed as a detailed design within the budget set.

The selected concept presents substantial curved forms within the landscape that are bold in a visual sense while retaining a minimal, understated, and elegant physical presence that complements the historical setting.

Figure 5.4 Architectural render of the bridge

courtesy of Pedelta Canada Inc.

5.6 Design details

The bridges incorporate high quality, durable, natural finish materials throughout, highlighted by the stainless steel components, and complemented by contrasting materials including wood, weathering steel and stone. When choosing all materials the life-cycle cost, which includes all anticipated maintenance costs, was considered at the design stage.

Solutions with two ribs either inclined outwards or inwards were explored as they define a visually attractive enclosed space. However, for a narrow bridge and an arch rise of around 9 m, these options would result in a bulky design with disproportionate dimensions. Also, skew views of the bridge would be rather cluttered visually, as typically happens with the visual crossing of cable-stayed bridges with two planes of stays in a fan arrangement. A single arch rib bridge results in a clean and well-defined form. It also provides open views on one of the sides. Placing the arch centred with the deck would have created a similar expression but it would become a physical barrier for users.

The selected solution included a tied stainless steel network arch with a distinctive crossing diagonal hanger pattern and a triangular cross-section profile. The arch was inclined at 18° to provide a more transparent and elegant structure. The arches tilt in opposite directions for each bridge to create a more dynamic visual appearance.

Rotating the arches means that the permanent and pedestrian loads create torsion on the deck. The inclination of the arch at an angle (Ω) can be translated to an increase of the axial forces in the hangers and the arch rib by a factor of 1/cos(Ω), i.e. for an angle of 18° this is approximately a 5% increase compared to an arch in a vertical plane. The structural system selected for both bridges is similar, with a slightly different geometry.

The north bridge has a single span with a total length of 52 m between the axes of the abutments. The arch has a parabolic elevation with a maximum rise over the deck elevation of 9 m resulting in a dynamic and relatively flat rise-to-span ratio of 1:5.8. The hollow rib has a triangular cross-section 900 mm wide and 450 mm deep with a central web made from steel plates with thicknesses ranging between 15 mm and 40 mm. A triangular cross-section was selected to enhance its visual slenderness, as well as to facilitate fabrication utilizing standard hot-rolled steel plates.

Figure 5.5 North bridge looking south

courtesy of Pedelta Canada Inc.

The arch is connected to the tie girder at both ends and two families of inclined hangers that cross each other once. The hangers are inclined at 60° to the horizontal and consist of 36 mm diameter stainless steel rods. The threaded fork ends of the rods facilitate length adjustment. The forks are connected to both the arch rib and the deck with gusset steel plates forming a pinned connection. The arch system with the inclined hangers results in a structurally efficient configuration that works like a truss with lower bending moments and shear forces, even for asymmetrical live loads, compared to arches with vertical hangers. The chosen pattern increases the lateral and vertical structural stiffness as well as the buckling capacity of the arch rib. Therefore, the cross-sections for both the arch and the tied girder were smaller, reducing the overall weight of the bridge and the costs associated with the materials and the foundations.

The deck system is finished with a 180 mm deep concrete slab on top. The slab is reinforced with stainless steel rebars and acts compositely with the box girder and ribs. Composite action was achieved by welded headed shear studs made from 1.4404

(316L) austenitic stainless steel. The studs are 22 mm in diameter and 120 mm high and were automatically welded. Unlike other lighter deck systems, the concrete deck has a lower mass and at the same time it provides a higher damping ratio required to prevent excessive vibrations that could otherwise be uncomfortable to the users.

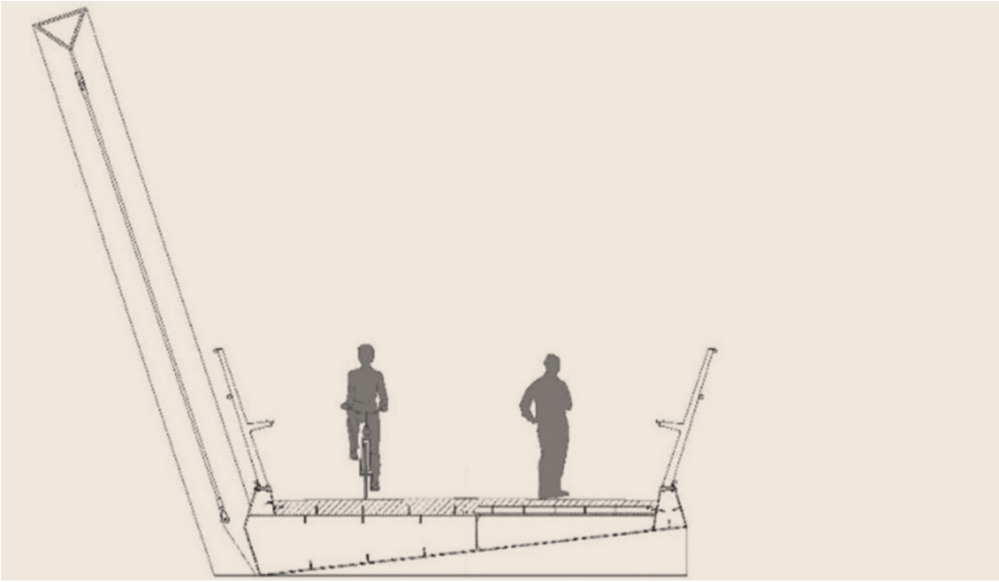


Figure 5.6
Typical cross-section of the North Bridge

courtesy of Pedelta Canada Inc

Unlike the North Bridge, the south crossing links the Ordnance Triangle to Fort York with a 5 m elevation difference that imposes a different bridge design concept. After assessing various arch alternatives, the solution that best fits the site constraints was a single span arch connected to a V-shaped pier on the south end. This unusual structural system is very efficient as it transforms the thrust of the arch into a set of axial forces in the V-pier, and it provides the necessary clearance underneath the crossing.

Similar to the North Bridge, the arch is inclined at 18° to the vertical, but in this case, it tilts towards the west to open up views towards the downtown skyline. It uses the same arch and tie box girder configuration as for the North Bridge with some adjustments to the arch width and altered plate thicknesses to adapt to the different structural demands.

The South Bridge landing included a 58 m long structural ramp on the west side terminating in a cantilevered lookout on the east side. The ramp is a continuous reinforced concrete girder with a span of 12 m to minimize the structure depth and open views underneath. The structure is continuous with the bridge and integral with the pier to reduce the need for future maintenance. The piers have a trapezoidal cross-section and are made of concrete. Two side faces of each pier are clad in weathering steel to provide a natural material contrast with the stainless steel that helps visually ground the bridge in its heritage setting.

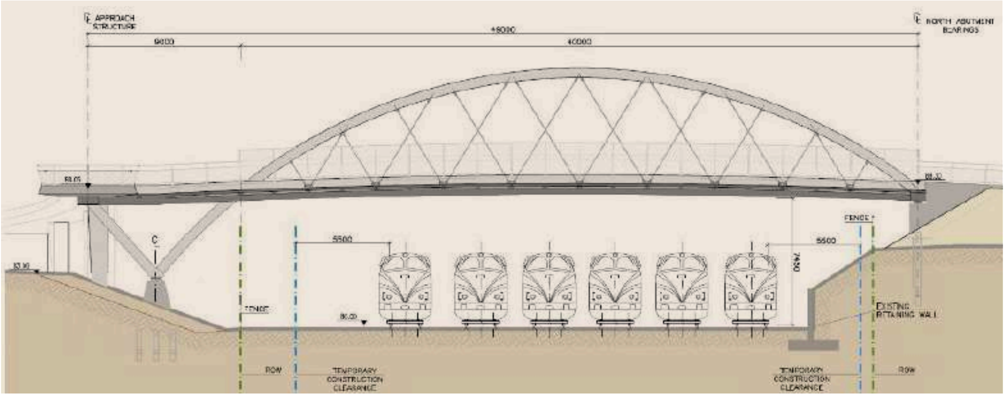


Figure 5.7 South bridge elevation

courtesy of Pedelta Canada Inc.

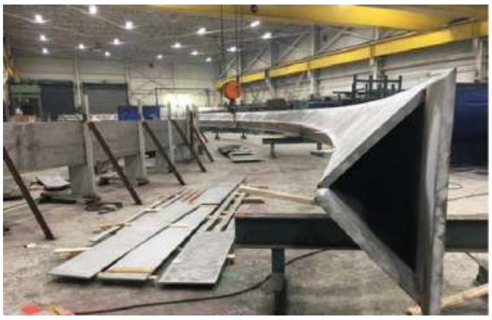
5.7 Superstructure construction

Due to the presence of softened clay soils, shallow spread footing foundations were not suitable for support of the pedestrian bridge abutments, and deep foundations have been adopted. Shallow foundations were also assessed for the more lightly loaded piers in the central and western portions of the sloped pathway on the Fort York area; however, deep foundations were selected for support of these portions of the structure to enhance the performance and avoid any differential settlement. The deep foundation solution consists of steel H-piles, fitted with bearing points and driven into the shale. The abutment and pier pile caps have been maintained as high as possible, to minimize excavation and groundwater control requirements. The project includes an innovative, flexible and attractive precast retaining wall with a reinforced soil system to retain the south and east faces of the North Landing and abutments. Face slopes are angled back slightly to provide a naturalized terraced stone effect that will blend well into the new South Stanley landscape and contrast effectively with the cast-in-place concrete bridge abutment.

Fabrication and erection were carried out in accordance with the AISC Design Guide 27 Structural Stainless Steel (DG 27). Welding was performed in accordance with the American Welding Society (AWS) D1.6/D1.6M. Stainless steel is, in many respects, different from carbon steel and should be treated accordingly. It is crucial to preserve the good surface appearance of the stainless steel surfaces throughout fabrication with simple precautions and good engineering practice. Greater care is required when storing and handling stainless steel than carbon steel to prevent damaging the surface finish and to avoid contamination by carbon steel and iron. Stainless steel can be cut by most of the usual methods, but power requirements are greater than those used for carbon steel due to work hardening. Grade 1.4462 duplex stainless steel has excellent machining properties compared to other stainless steels. It has good weldability and most of the typical welding methods such as Shielded Metal Arc Welding (SMAW), Gas Tungsten Arc Welding (GTAW), Gas Metal Arc Welding (GMAW), Submerged-arc Welding (SAW) among others can be used. The material should be welded without preheating and allowed to cool between welding passes to temperatures below 150ºC.

Filler materials shall be used. Post-weld heat treatment after welding with filler is not necessary. Inspection of welds was carried out by AWS certified weld inspectors, experienced in welding stainless steel. Examination methods for welds are like those used for carbon steel. In order to restore the corrosion resistance, it is necessary to conduct a post-weld treatment such as pickling and brushing.

Figure 5.8
Steel fabrication of the box girder and the arches



courtesy of Mariani Metal Fabricators

The steel bridges were fabricated in a steelyard in the Toronto metropolitan area. Both the tied girder and arch were fabricated to the required camber to compensate for deflections due to dead loads and match the design profile elevation. All visible stainless steel surfaces were bead blasted after pickling to get a consistent uniform dull finish with a natural silver colour and remove all scale and surface contamination arising from fabrication. They were then transported in segments and assembled close to the final position of the bridge. They were lifted by crane on temporary support towers and after the final assembly of the steel structure, the hangers were installed and hand tightened. After the installation of the hangers, the temporary supports were removed and the bridges were lifted into their final place. The bridges were erected in just a few hours during a single night possession (for each bridge) to minimize rail disruption, using one crawler crane.

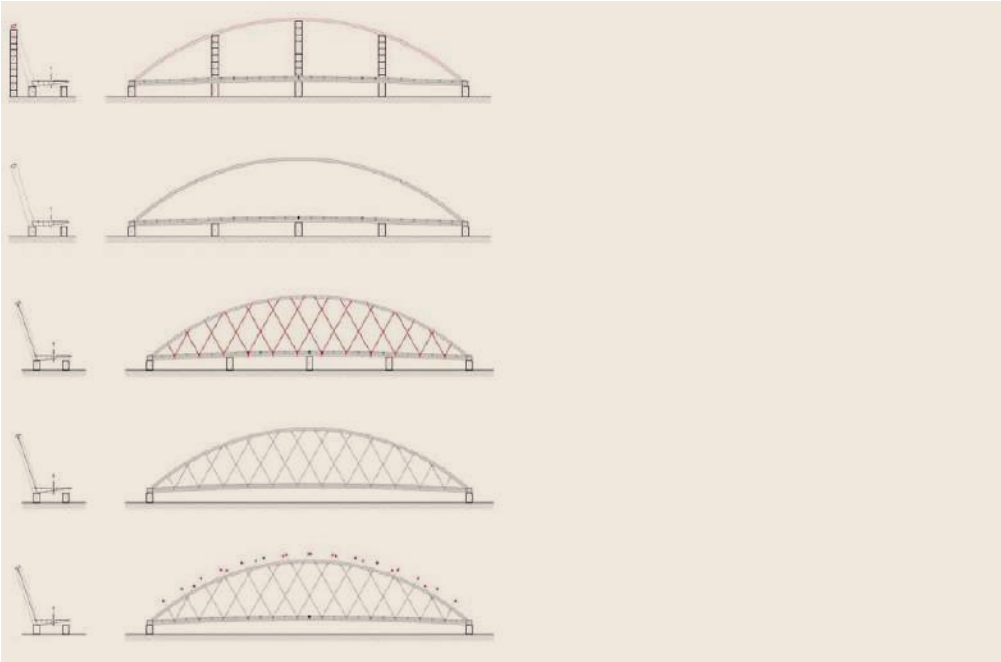


Figure 5.9
Bridge erection sequence

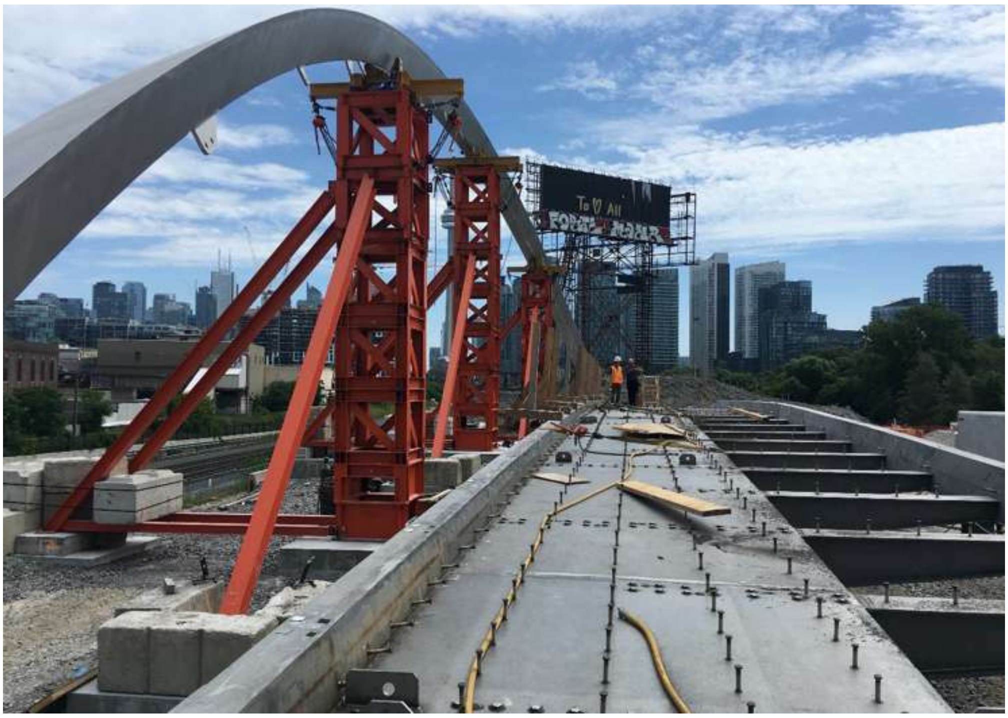


Figure 5.10 Bridge during construction

courtesy of Pedelta Canada Inc.

5.8 Feedback

The bridge was given the Award of Excellence for the bridge category at the 2019 Ontario Steel Design Awards by the Canadian Institute of Steel Construction. It was also given the Award of Excellence at the 2020 Canadian Consulting Engineering Awards by the Association of Canadian Consulting Engineers. For the latter it was described as a 'social device' that promotes sustainability as well as emphasizing the cultural heritage of the city. The innovative use of duplex stainless steel for the entirety of the bridge was applauded by the jury as an excellent technical innovation for long-term durability in Canada.

The use of stainless steel for the entire superstructure, for the first time in a bridge project in North America, is the highlight of the bridges. Stainless steel enhances the visual appearance of the bridges, and of course the higher strength and ductility and most importantly its superior corrosion resistance are true benefits. From the life cycle cost analysis carried out for the specific project, it was found that the initial investment (approximately 12% increase of the overall cost due to the use of stainless steel) would be balanced by the significantly lower maintenance requirements, and the overall cost of ownership would be lower still compared to more conventional solutions.


Pooley Bridge, UK
Made of Outokumpu's lean duplex grade Forta LDX 2101
The lower nickel content of Forta LDX 2101 means lower costs than with other types of stainless steel. Another factor in favour of duplex is its aesthetic appeal and corrosion resistance, which means the bridge will not need any painting. This reduces life-cycle costs and protects the environment.
Outokumpu is a leading producer of sustainable stainless steel with recycled content of over 90% and carbon footprint 75% lower than the global industry average.
Learn more about the benefits of stainless steel:
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sustainable stainless steel

POOLEY BRIDGE

6.1 Facts and figures

Owner / Client:	Cumbria County Council
Consulting Engineer:	GHD, Mott MacDonald
Architect:	Knight Architects
Main Contractor:	Eric Wright Civil Engineering
Steel Tonnage:	110 tonnes of stainless steel (1.4162 and 1.4404)
Steel Manufacturer:	Outokumpu
Steelwork Fabricator:	WEC Group
Fabrication Commenced:	2019
Public opening:	October 2020
Further Information:	https://www.knightarchitects.co.uk

Figure 6.1 The Pooley New Bridge

6.2 Design basis

Design Standards:	BS EN 1993, BS EN 1994 and BS EN 1997 with corresponding UK National Annexes. BS EN 10088-4:2009 and BS EN 206:2013(+A1:2016) for materials specification
Loading:	BS EN 1990 and BS EN 1991 with corresponding UK National Annexes
Design Life:	100 years

6.3 Location

The bridge is located in Pooley Bridge in the north-western English county of Cumbria, near the Lake District National Park, a UNESCO World Heritage Site with beautiful landscape which makes it a popular tourist attraction.

Figure 6.2 Aerial view of the bridge during construction

courtesy of Cumbria County Council & Eric Wright.

6.4 Bridge context

The new bridge replaced the original historic Grade-II listed stone arch bridge which collapsed during flooding in 2015. The need for a replacement bridge was urgent to avoid a 16 km long detour.

The desires of the community, the Lakeland-setting and its future uses all played an important role in determining the scope for the eventual structure.

A meaningful stakeholder engagement process took place, listening to the community's views and focusing on their common aspirations for a design that fitted both their sense of identity and location, whilst satisfying relevant technical standards.

The project required outstanding teamwork and meticulous planning to counter several challenges. The site itself posed challenges, being situated on a mountainside with poor ground. In-river work had to be factored in, as well as the peak tourist season and salmon spawning times, all of which limited the available timeframe for work to take place.

6.5 Bridge conceptual design

Emphasis was given to conceive a flood-resilient and future-proof bridge, complying with the current technical standards and the Environment Agency (EA) regulations.

Therefore, a single clear span was chosen as it minimises environmental impact and flood risk.

Unusually, an integrated community engagement and concept design process was led by a specialist Bridge Architect and the resulting bridge design by Knight Architects is a slender 40 m span open-spandrel arch with an innovative composite stainless steel and high strength concrete structure. The use of lean duplex stainless steel made it possible to deliver a bridge that was cost effective, looks contemporary, will age naturally like the previous historic bridge, has excellent durability without the need for maintenance, and has about 25% more structural capacity than a conventional steel bridge.

Duplex stainless steel has also allowed the bridge to be light, both in terms of weight (to facilitate construction) and in terms of perceived slenderness. The choice of material was also based on whole life cost, having been considered by the client as a cost-effective solution when taking maintenance savings into account.

6.6 Design details

The bridge features a slender composite deck with a steel plate at the bottom, which resists tension forces and provided temporary support to the wet concrete during construction. The deck thickness was minimised through the provision of shear connectors, welded to the plate, to ensure composite action between the steel plate and the concrete. Stainless steel T-sections, fillet welded to the bottom steel plate were used, instead of the traditional headed studs. The use of stainless steel studs was also considered, however, due to supply issues and long lead times, fabricating the connectors from plate was the preferred option.

Duplex stainless steel grade 1.4162 (commonly known as LDX2101®) was used for the main structure, the shear connectors and the handrails. Bearing plates and lifting points were from austenitic stainless steel 1.4404 (316L). The fabrication included some complex details, and generally a lot of labour hours were used mainly due to the large quantity of welded details. However, the slender design, made possible by the high strength duplex stainless steel, minimised the amount of material used and its associated embodied CO_2 content. The specific type of steel used has one-fifth of the embodied carbon of the global average of stainless steel, due to its over 85% recycled content and the low-carbon energy used at production sites.

The bridge has 7.5 m-long hidden back-spans within the abutments to transfer the horizontal component of the arch compression to the deck. This provides a traditional deck-arch appearance, but without transferring horizontal reactions to the low-capacity ground conditions.

6.7 Superstructure construction

When it came to the construction programme, a number of challenges had to be overcome. The construction of the bridge was constrained by environmental and economic aspects as the work had to happen outside of the salmon spawning season, but without impacting on tourism.

These issues left a very short window for the onsite works and encouraged maximising offsite construction. Using steel as part of the structure was fundamental to achieving these requirements. The 110 tonnes of steelwork, all made up of bespoke sections, was fabricated in four quarters, taking approximately 22,000 man-hours to complete.

Figure 6.3 Bridge during construction

courtesy of WEC & Eric Wright.

A 1,350 tonne mobile crane was used to lift the 290 tonne main span in place. Additional challenges were the transportation of parts through narrow roads and the onsite assembly in a limited working area. After the lift, the back-spans were installed inside the abutments, the temporary ties linking the arch springings were removed, and the concrete part of the composite deck was poured to complete the permanent structural system.

Figure 6.4
New Pooley bridge being lifted in place

courtesy of Eric Wright.

6.8 Feedback

The bridge was awarded in the Structural Steel Design Awards (SSDA) of 2021 with the judges stating that ingenuity, innovation and beauty have been combined in this remarkable replacement bridge, which is the UK's first structural stainless steel road bridge. Importantly, the new bridge pays homage to its predecessor and other examples of British bridge heritage. It also looks to the future, becoming a fitting addition to the site thanks to its lightness and transparency, not only providing unhindered views but minimising obstruction to water in flood events.

The Pooley Bridge design and construction is an extraordinary example of the community, the designers and the contractors working in harmony to provide an outstanding bridge which serves the community and respects the protected landscape setting.

The replacement bridge was open for use on 23 October 2020 and is now attracting visitors to the area in its own right.

WIRKOWICE ROAD BRIDGE

7.1 Facts and figures

Owner / Client:	Zarząd Dróg Powiatowych (ZDP) Krasnystaw
Consulting Engineer:	Fasys Mosty Sp. z o.o. Wrocław
Architect:	-
Main Contractor:	Mosty Zamość - Tomasz Czyż
Steel Sub-Contractor:	ArcelorMittal Steligence® Fabrication Centre
Public opening:	October 2020
Steel Tonnage:	62 tonnes*
Protective Treatment:	Weathering Steel
Further Information:	https://www.arcelormittal.com

* accounts for finished main girders only

Figure 7.1 Road bridge in Wirkowice - first bridge in Europe made of S460J2W weathering steel sections

courtesy of Mosty Zamość - Tomasz Czyż

7.2 Design basis

Design Standards:	Polish Standard PN-85/S-10030
Loading:	Class B according to PN-85/S-10030
Design Life:	100 years

7.3 Location

The bridge is located in Wirkowice in the Lubelskie (Lublin) province of Poland over the Wieprz River along the district road No. 3173L between Wirkowice and Tarzymiechy.

7.4 Bridge context

Completed in October 2020, the Wirkowice bridge in Poland is the first road bridge in Europe to be constructed with S460J2W weathering steel sections. It was built with Arcorox® beams, produced by ArcelorMittal Europe – Long Products.

The Wirkowice composite bridge is the result of the 'Wieprz River road bridge reconstruction on 3142L district road' investment project. The project was completed well ahead of schedule, allowing the local residents to use the new crossing as quickly as possible.

7.5 Bridge conceptual design

The bridge was originally designed as a continuous, three-span steel-concrete composite structure. The main structure consisted of four rolled HL 1000A girders in S355J2+M grade, and the steel crossbeams are HL 1000A in S355J2+M grade. The basic parameters on which the design was based are:

- Road class Z (speeds of up to 60 km/h)
- Load class B according to PN-85 / S 10030
- Theoretical spans: 20.75 m x 18 m x 17.75 m
- Total width: 11.2 m
- Skew angle: 90 degrees

As part of the design optimisation, a composite steel-concrete structure using rolled profiles in the new S460J2W+M steel grade (according to EN 10025-5: 2019) was proposed. HEA 900 sections were adopted, allowing for weight reduction and an increase in bridge clearance. The S460J2W steel grade is characterised by a high yield strength of 460MPa and an increased resistance to atmospheric corrosion. Concrete class and reinforcing steel were C35/45 and B500SP, respectively.

7.6 Design details

The continuous three-span (20.75 m + 18.00 m + 17.75 m) superstructure accommodates two traffic lanes, having a width of 3 m plus 0.5 m strips and 1.5 m sidewalks on both sides. The transverse spacing of the main girders is 2.80 m. They were connected to the 210 mm thick C35/45 concrete deck slab with 22 mm diameter, 150 mm high shear studs. The deck slab and the supporting cross-beams were made of in-situ concrete cast on the formwork which was supported by the steel girders. At the time of concreting, each of the spans was simply supported. During concreting of the deck slab, the beams were restrained by a temporary bracing system bolted to the webs of the main girders, which was subsequently dismantled at the same time as removal of the suspended formwork.

Figure 7.2 Underside view of road bridge in Wirkowice

courtesy of W. Ochojski from ArcelorMittal, Poland

7.7 Superstructure construction

The pillars were constructed as monolithic concrete walls, supported on existing concrete piles. Abutments were re-built, and supported on the existing set of piles. Before installing the steel structure, the concrete cross-beams were partially concreted up to the level of the lower surface of the bottom flange of the rolled girders, so that the beams could be placed directly on the cross-beams, without the need for any temporary structures. Each partial concrete cross-beam was placed on the end set of bearings (4 No. bearings), and it was restrained against rotation during the erection of the steel beams. The steel sections were placed onto the cross-beams span-after-span, in full span lengths, to avoid any site welding. Beams were acting as simply supported in each span, carrying all the self-weight, weight of the formwork, reinforcement and wet concrete. To prevent buckling of the steel beams, a temporary bracing system was provided with bolted connections to the webs of the girders. Formwork for the concrete deck was supported by the steel girders. The rolled sections were designed and manufactured with pre-cambering, based on the predicted deflection of the girders under both static and dynamic loads. Concreting of the deck was done at the same time as that of the remaining parts of the concrete cross-beams, from the bottom flange of the rolled beams up to the top level of the deck. Final road and pavement layers, together with the bridge equipment, were installed after the deck was completed.

Figure 7.3 Wirkowice road bridge during construction

courtesy of W. Ochojski from ArcelorMittal, Poland

7.8 Feedback

The adopted design solution allowed the main contractor of the project to simplify and speed up the construction process. Beams were delivered in full span lengths, therefore the assembly was simplified by eliminating any site welding. The introduction of the concrete cross-beams did not impact the size and construction of the abutments and the pillars. This is because the use of high strength weathering steel significantly reduced the weight of the steel superstructure to compensate for the additional weight due to the cross-beams.

The owner also appreciated the maintenance-free structure, mainly due to the use of weathering steel, as well as the significantly earlier completion than initially foreseen.

BISKUPICE ROAD BRIDGE

8.1 Facts and figures

Owner / Client:	Zarząd Dróg Powiatowych (ZDP) Świdnik
Consulting Engineer:	Fasys Mosty Sp. z o.o. Wrocław
Architect:	-
Main Contractor:	Dura Sp. z o.o. Lublin
Steel Sub-contractor:	ArcelorMittal Steligence® Fabrication Centre & Mostostal Puławy S.A.
Public opening:	March 2021
Steel Tonnage:	16 tonnes*
Protective Treatment:	Weathering Steel
Further Information:	https://www.arcelormittal.com

* accounts for finished main girders only

Figure 8.1 Biskupice road bridge - the first hybrid bridge in Europe made of S460J2W steel

courtesy of A. Stempniewicz

8.2 Design basis

Design Standards:	Polish Standard PN-85/S-10030
Loading:	Class B according to PN-85/S-10030
Design Life:	100 years

8.3 Location

The bridge is located in the village of Biskupice in Poland, crossing the Giełczew River.

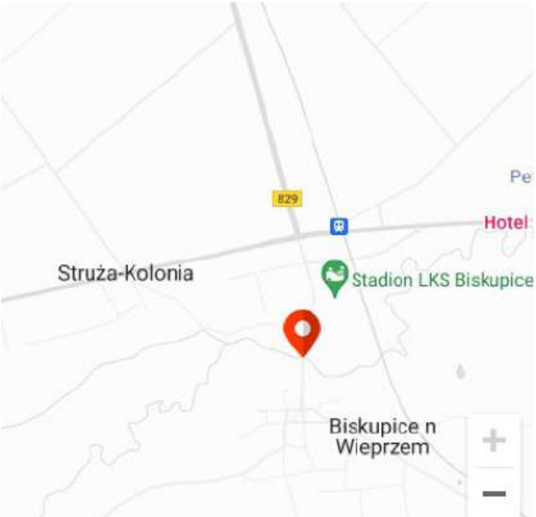

Figure 8.2
Map showing location of bridge

8.4 Bridge context

The new bridge replacing the existing one was completed in June 2021, and it is the first composite bridge in Europe combining the innovative PreCoBeam (Prefabricated Composite Beam) technology with weathering steel grade S460J2W sections.

8.5 Bridge conceptual design

Alternative solutions including the use of conventional composite girders were considered. In choosing between them, the main objective was to minimise the weight and at the same time enhance the strength of the structure, particularly the fatigue strength, which is one of the reasons the PreCoBeam solution was chosen.

8.6 Design details

The bridge is continuous with two equal spans of 15.25 m. The 11.1 m wide deck carries two roadway lanes (2 x 3 m each), a 1.8 m wide pavement on one side, safety barriers, and stairs that allow for safe service access under the structure. The deck superstructure is formed by a steel-concrete composite section. However, the structural steel part of this cross-section is not connected with the intermediate support, it ends in the so-called transition zone. The cross-section over the intermediate support is a reinforced concrete slab.

The transverse spacing of the main girders of the structure is 2.78 m. These hybrid girders were made of HEB900 steel sections cut in half to form T-sections. The 13.1 m sections were oxy-fuel cut along the web according to a defined geometry. The resulting shape provides an efficient connection to the 250 mm thick deck slab by means of composite dowels, eliminating the need for stud welding and hence optimising the volume of works by adopting a fully automated process. In the transition area, the steel beams were shaped in such a way as to maintain the required smooth change in stiffness and thus the gradual increase in internal forces in the load-bearing elements. The bridge slab was made of in-situ C30/37 concrete cast on formwork supported by a temporary structure, with transverse steel beams supported on re-used IPN550 beams obtained from the demolition of the existing bridge.

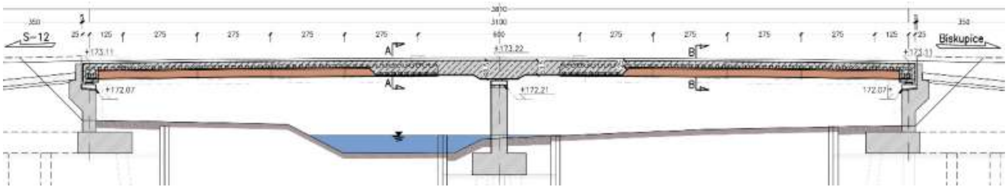

Figure 8.3 Longitudinal section of Biskupice road bridge

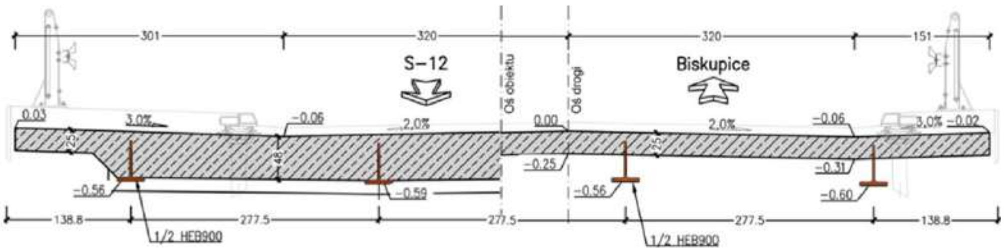

Figure 8.4 Cross-section of Biskupice road bridge

The high degree of prefabrication of the elements translated into shorter assembly times on site. The adopted PreCoBeam solution optimised tonnage (leading to a 30% material reduction), ensured faster delivery times and is expected to decrease maintenance costs. In addition, construction depth was minimised which was particularly important due to site restrictions.

Figure 8.5 Innovative PreCoBeam technology for the bridge S460J2W girders

courtesy of A. Stempniewicz

8.7 Superstructure construction

The bridge superstructure was designed as a continuous double span, with bearings over the abutments and a middle pillar. The abutments and pillar were constructed as monolithic concrete elements, supported on tubular concrete piles due to the poor soil conditions. The unusual composite superstructure was realised on site, by using a temporary structure supporting the formwork, the reinforcement, as well as the steel sections during the construction phase. Temporary supporting elements were located near the pillars to avoid any obstruction / interference with the river flow underneath. The steel sections were delivered already cut to the right length, ready to be erected. Asphalt layers, along with the pavement and equipment were placed after the deck was completed and all temporary structures were removed.

Figure 8.6 Biskupice road bridge during construction

courtesy of Dura Sp. z o.o.

8.8 Feedback

The innovative design optimised material use and reduced on-site construction time, and the use of weathering steel is expected to reduce future maintenance costs considerably. These aspects were highly appreciated by the client. Another challenge was to limit the depth of the superstructure to maintain an adequate clearance underneath the bridge, which was achieved by the implemented solution.

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SPANNING TO THE FUTURE – STRUCTURED IN STEEL

This publication was prepared by SCI's Steel Bridge Group and presents case studies where steel was used effectively and efficiently in bridge construction, to best exploit the benefits of the material. The case studies presented illustrate creative collaborations between engineers, architects, contractors and their clients in service of society. It is hoped that they will inspire and inform planners and designers to always consider the use of steel during the initial planning phase of a project, and to use steel when appropriate.

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P418 | Completion of appendix 18/1

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