e-mosty

ISSUE 02/2026 JUNE

BRIDGES IN AUSTRALIA AND NEW ZEALAND

LIST OF CONTENTS

Front Cover: *Kangaroo Point Bridge* ***Credit:*** *WSP*
Back Cover: *Molonglo River Bridge, Canberra* ***Credit:*** *BMD Constructions*

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Dear Readers

This special edition of the e-mosty magazine is dedicated to bridges in Australia and New Zealand. I visited most of them during my trip last November, and I also met great bridge designers, engineers, and others who contribute to infrastructure development in both countries. Thank you all for welcoming me, showing me your projects and for your cooperation.

Now we have the pleasure to bring articles about the following projects:

- *Kangaroo Point Bridge, Brisbane*
- *Boorloo Bridge, Perth*
- *Molonglo River Bridge, Canberra*
- *Fauna Bridge on the Wadandi Highway, Bunbury*

All of them are in Australia.

We also present the Manganui Gorge Suspension Bridge in New Zealand.

I want to thank all authors, people, and companies for their cooperation, and our Editorial Board, especially Ken Wheeler and Thomas Cooper, for their assistance with this edition and for reviewing the articles.

We also thank **our partners** *for their continuous support.*

In future e-mosty and e-BrIM editions, we are going to publish more articles from Australia and New Zealand, including the Fremantle Bridge (Swan River Crossing), which is currently under construction.

On behalf of the organisers, we would like to invite you to the following conferences:

- ***13th International Conference on Bridge Maintenance, Safety and Management IABMAS****, which will be held from* ***7th to 9th July 2026 in Orlando, Florida, USA****. More information about the conference is on pages 10 and 11 and at https://iabmas2026.org.*
- ***2026 World Bridge Engineering Conference****, with an emphasis on Innovative Bridge Technologies and Accelerated Bridge Construction, which will take place on* ***1st and 2nd December 2026 in Miami, Florida, USA****. More information is on page 12.*

The next e-mosty will be published on 20th September. The next e-BrIM will be released on 20th October in both **English** *and* **Spanish***.*

We welcome your articles for both the e-BrIM and e-mosty magazines. You can contact me at magda@e-mosty.cz.

Magdaléna Sobotková

Chief Editor

INTERNATIONAL ONLINE PEER-REVIEWED MAGAZINE ABOUT BRIDGES

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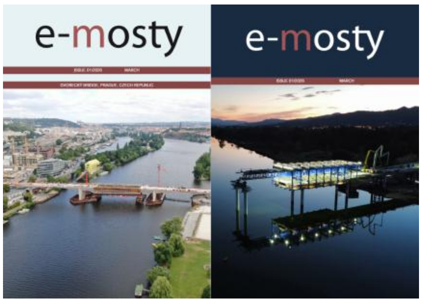

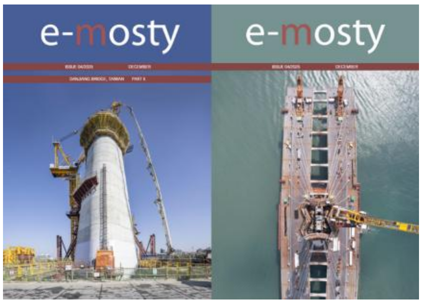

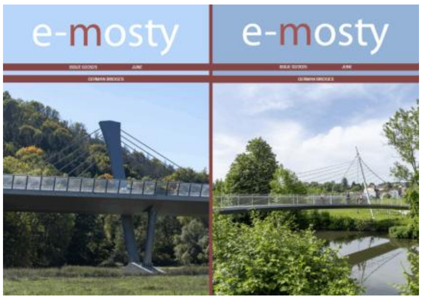

The magazine **e-mosty** (“e-bridges”)
is an international, interactive,
peer-reviewed magazine about bridges.

It is published at www.e-mosty.cz and can be read
free of charge (open access)
with the possibility to subscribe.

It is published quarterly: 20 March, 20 June,
20 September and 20 December.
The magazines stay **available online**
on our website as pdf.

The magazine **brings original articles about bridges
and bridge engineers** from around the world.
Its electronic form enables the publishing
of high-quality photos, videos, drawings, links, etc.

We aim to include **all important and technical
information** and show the grace and beauty
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We are happy to provide media support for important
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Our Editorial Board comprises bridge engineers
and experts mainly from the UK, US and Australia.

The readers are mainly bridge engineers, designers,
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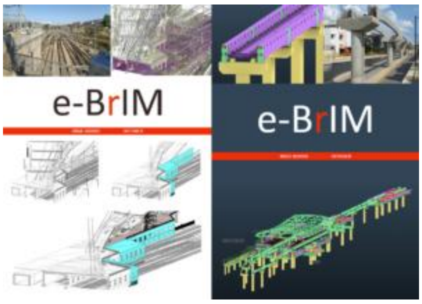

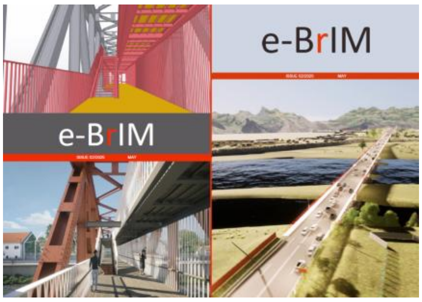

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It is typically published three times a year:
20 February, 20 May and 20 October.

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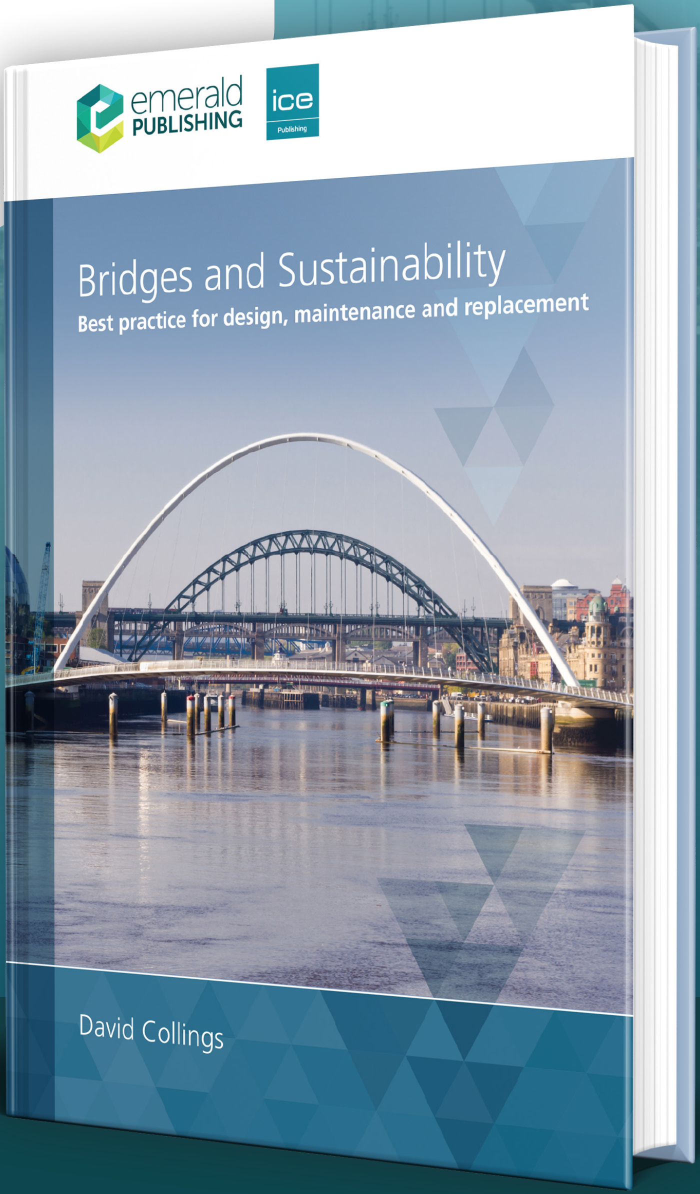
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IABMAS 2026

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Welcome to IABMAS 2026 in Orlando, Florida, known as the City Beautiful! The conference theme, "Smart and Sustainable Bridges for the Future," will explore innovative solutions that will re-shape the future of bridge engineering. The objectives of IABMAS 2026 are to address all aspects of bridge maintenance, safety, risk, economic issues and management. Specific emphasis will be on bridge repair and rehabilitation issues, bridge management systems, the needs of bridge owners, financial planning, life-cycle evaluation costing and investment for the future.

The implications and applications of artificial intelligence, digital twins and robotics in the management of existing bridge stocks are also among the relevant objectives. IABMAS 2026 aims to act as a forum for academics, practitioners, owners and operators to discuss recent advances and identify future research directions. Together, we will share knowledge, inspire progress, and strengthen connections within our vibrant international community. We look forward to welcoming you in Orlando in July 2026.

Learn more at iabmas2026.org

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IABMAS

Welcome Reception

July 6, 2026

Conference

July 7-9, 2026

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July 10, 2026

For other key dates please visit our website.

Venue/Accommodations

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WELCOME

Welcome to IABMAS 2026 in Orlando, Florida, U.S.A., known as the City Beautiful! The conference theme, "Smart and Sustainable Bridges for the Future," to explore innovative solutions that will re-shape the future of bridge engineering. The objectives of IABMAS 2026 are to address all aspects of bridge maintenance, safety, and management. Specific emphasis will be on bridge repair and rehabilitation issues, bridge management systems, the needs of bridge owners, financial planning, life-cycle evaluation costing, and investment for the future. Additionally, the conference will focus on bridge-related safety, risk, and economic issues. The implications and applications of AI, Digital Twins, Robotics in the management of existing bridge stocks are also among the relevant objectives. IABMAS 2026 aims to act as a forum for academics, practitioners, owners, and operators to discuss recent advances and identify future research directions. Together, we will share knowledge, inspire progress, and strengthen connections within our vibrant international community. We look forward to welcoming you in Orlando in July 2026.

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Hae-Bum Andrew Yun

Chairs of IABMAS 2026 Conference, Orlando, Florida

KEY DATES

The conference dates will be July 7-9, 2026. The welcome reception will be on July 6, 2026, and the technical tours will be on July 10, 2026. For other key dates please visit our website.

GENERAL INFORMATION - VENUE, ACCOMODATION, CITY

The conference venue will be Rosen Plaza Hotel. The newly renovated hotel and its settings will enable easy access to sessions, comfortable and spontaneous networking and meeting opportunities, as well as relaxing spots for the attendees and accompanying persons. Located on International Drive, the hotel is walking distance to many attractions, restaurants, 5 minutes from Sea World, 15 minutes from Universal CityWalk and Walt Disney World. While you immerse yourself in the technical sessions, don't miss the opportunity to explore the wonders of Florida. Orlando, renowned for its world-class attractions and offers endless entertainment for all ages. Additionally, you can discover the unique natural beauty of Florida's diverse ecosystems with visits to the Everglades, pristine beaches, and the picturesque springs.

LOCAL HOSTS, SOCIAL PROGRAM AND TECHNICAL TOURS

University of Central Florida (UCF) and IABMAS USA National Group will host the conference locally. As local hosts, the (UCF) and IABMAS USA are dedicated to making this conference an informative, enjoyable, and remarkable experience. A memorable social program and technical tours will be arranged for the attendees. These will be posted on the conference website https://iabmas2026.org.

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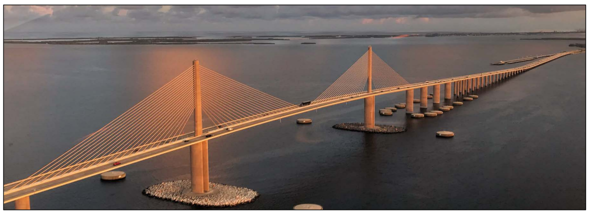
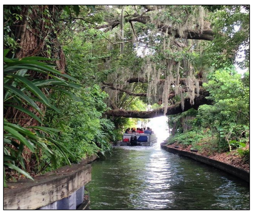
IABMAS USA





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What Bridge Engineers Should Know About AI

Nov 30, 1:00 PM – 4:00 PM

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KANGAROO POINT BRIDGE, BRISBANE, AUSTRALIA

BRIDGE AS DESTINATION

Susanne Bendsen

Dissing+Weitling

Figure 1: Kangaroo Point Bridge in Brisbane

THE BRIDGE

The architectural vision for Brisbane's new Kangaroo Point Bridge emerged from a careful response to its remarkable setting — the river, the skyline, and the everyday urban life unfolding along one of the city's most central waterfront locations.

Rather than treating the bridge as an isolated transport structure, the design contributes spatially, socially, and visually to the city itself. Its long-span cable-stayed structure and sensitive urban integration transform the river crossing into a public place and a new landmark for Brisbane.

The public response has been immediate. According to Brisbane City Council, the bridge already attracted more than 10,800 daily users shortly after opening in January 2025 — significantly exceeding the original projection of 6,100 daily users by 2036.

The intensity of use confirms the bridge's success as a fully integrated piece of public infrastructure, and – equally importantly – it shows that the bridge is used for far more than movement alone.

Pedestrians, cyclists and runners share the bridge with visitors who pause to take in panoramic views of the river and skyline, meet at cafés and restaurants, or simply spend time above the water.

The bridge has rapidly become recognised as both a must-visit destination and an identity-defining element within Brisbane's contemporary cityscape.

A GENEROUS STRUCTURE

Located between Brisbane CBD and Kangaroo Point, the bridge extends the city's pedestrian and cycling network. At the same time, it creates a sequence of public spaces above the water, along the river edge, and at the landing points. In this way, the project contributes more to the site than it occupies.

Through widened deck areas, viewing platforms and integrated public amenities, the bridge introduces new ways of experiencing the city and establishes new vantage points across the river and skyline.

STRONG IDENTITY — SLENDER EXPRESSION

A key architectural objective for Dissing+Weitling was to achieve visual lightness despite the considerable structural demands of the span.

The bridge deck is therefore articulated with a slim edge profile and carefully proportioned structural elements that reduce visual mass and reinforce the perception of horizontality across the river.

The asymmetrical cable-stayed composition establishes a distinctive silhouette on the Brisbane skyline while maintaining a slender and lightweight expression.

The sculptural mast rises as a singular vertical element that anchors the composition and gives the bridge a strong visual identity both from the river and from the surrounding city.

The detailing of the edge beams, cable anchorages, and mast interfaces contributes to a coherent architectural language in which technical components are expressed with precision and clarity rather than concealed.

The bridge alignment and geometry were equally important architectural parameters.

The gently curving approach spans create a dynamic movement sequence for pedestrians and cyclists, and the widening of selected deck areas creates opportunities for pause and social interaction.

Figure 2: View of the deck and the Bridge from water

Jesper Henriksen, Head of Bridge Design at Dissing+Weitling, explains:

"Although the design of Kangaroo Point Bridge has a strong visual identity, creating an iconic form has never been an objective in itself. A bridge becomes a landmark when it continues over time to provide value to both place and people through an elegant and well-integrated solution.

Our design philosophy is rooted in the Nordic design tradition, where resource optimisation and the human experience are fundamental principles. The speed with which Brisbane's citizens and visitors have embraced the bridge reflects the ability of the entire design team to approach a mobility project like this holistically."

INCLUSIVITY AND URBAN LIFE

The architectural strategy also focused on inclusivity and user diversity. The bridge accommodates a diverse range of users, and particular attention was given to movement flows, accessibility, visual openness, and the overall quality of the public experience.

Lighting design further reinforces the bridge's presence after dark, allowing the structure to remain an active and recognisable element within Brisbane's nighttime identity.

At the landing points, the bridge integrates seamlessly with surrounding public spaces, strengthening the relationship between the city and the riverfront. Hospitality venues and public amenities activate the waterfront and support vibrant year-round city life.

Kangaroo Point Bridge has rapidly become part of Brisbane's contemporary identity. Its success lies not only in the technical accomplishment of spanning the river with minimal intervention but also in its ability to create public value through architecture.

Figure 3: View of the complete Kangaroo Point Bridge

KANGAROO POINT BRIDGE

THE FUTURE OF TRANSPORT IN BRISBANE

Thomas Cooper, WSP, Denver, Colorado, USA
Claudia Love, Andrew Gallagher; WSP Brisbane, Australia

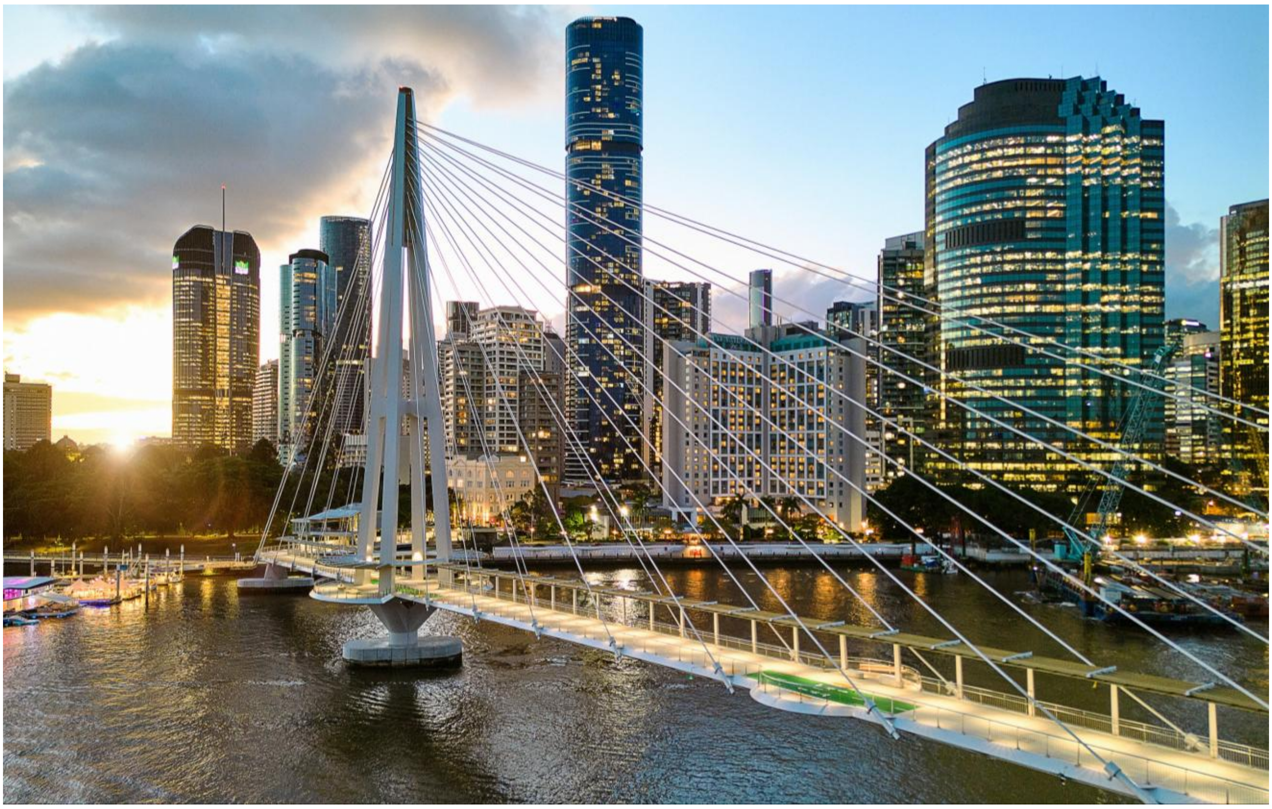


Figure 1: The completed bridge minimises impact to the Brisbane River while respecting the existing skyline in the Central Business District

INTRODUCTION

The Kangaroo Point Bridge is a transformative addition to Brisbane's urban landscape, redefining the way people move between the city centre and eastern suburbs.

Focused on the future of transportation, it is part of a Brisbane City Council program to promote pedestrian, cycle, and e-mobility transport across the city. The bridge was opened in December 2024, delivered by a consortium led by BESIX Watpac.

Crossing the Brisbane River, the 460m-long bridge provides a striking, elegant form, connecting two active areas of the city: the densely residential suburb of Kangaroo Point and the centre of the Central Business District (CBD) via the Botanic Gardens. With a 1.2m deep composite steel box superstructure and 95m tall steel box tower, the 183m long asymmetric cable-stayed main span, paired with the narrow width required for a pedestrian and cycle bridge, is one of the longest of its kind internationally.

The objective of this article is to discuss design and construction challenges and highlights encountered in undertaking this ambitious pedestrian and cycling crossing that's quickly become Brisbane's latest icon.

HISTORY

Options to connect Kangaroo Point and the Brisbane CBD have been studied since 1860. With Brisbane moving toward more sustainable forms of transport, the final product (Kangaroo Point Bridge) focuses on accessibility and reducing congestion on other bridges, while activating key areas along and adjacent to the bridge alignment.

The design features pause points at intervals along the bridge, an above-water restaurant/bar (Stilts Dining) and a riverside café (Mulga Bill's) on the adjacent plaza area near the Botanic Gardens side of the river. More than 'just a bridge', the iconic structure is fast becoming a must-visit destination for Brisbane's residents and visitors.

The project's attention to urban activation establishes it as a new standard for footbridge vitality, enhancing opportunities for dining, events, social connection and wellbeing on either side of the river and reactivating the precincts at the bridge's landing points.

PERFORMANCE SPECIFICATION

The Kangaroo Point Bridge preliminary and detailed designs were delivered under a Design and Construct contract following a dual "Early Contractor Involvement (ECI)" process. The ECI stage further developed the Concept Design, via an interactive process with Council and their advisors.

The project scope and technical requirements formed part of the contract documents, with design specified to comply with Australian Standard AS5100-2017 Bridge Design and international standards as applicable. International standards were adopted to address issues specific to long span cable-stayed pedestrian bridges, including cable stay performance, wind performance and pedestrian comfort from the lightweight bridge footfall dynamic response.

Key international standards/guidelines referred to for the design included SETRA (2006), Technical Guide: Footbridges – Assessment of Vibrational Behaviour of Footbridges Under Pedestrian Loading, and Fib Bulletin 89 Acceptance of Stay Cable Systems Using Prestressing Steels, 2019.

Figure 2: Location of the Bridge
Click on the image to open Google Maps

Client and Owner:	Brisbane City Council
Principal Contractor:	BESIX Watpac
Architecture and Design:	Blight Rayner Architecture, Dissing + Weitling, ASPECT Studios, Right Angle Studio, Blaklash, UAP
Communication and Stakeholder:	Rowland
Concept and Reference Design:	Brisbane City Council with Arup and COX Architecture
Embedded Specialists:	Rizzani de Eccher, Tensa
Engineering:	WSP, RWDI

DESIGN

Concept Design

Paying homage to the existing environment, the Kangaroo Point Bridge is designed to accommodate the existing skyline of Brisbane. While the central mast is intended to be a focal point, the design respects both the existing skyline, the urban connections and the natural environment along the riverbank.

The scale and nature of an iconic bridge crossing a river tend to include large structural elements, but the bridge must also address the perspective of the user. This need led to the development of a four-legged tower, which balances structural requirements while creating an unobstructed pedestrian and cyclist flow along the bridge.

To enable a safe and sustainable shift toward active transport modes, the 6.8m width of the bridge allows dedicated paths for pedestrians and cyclists, by line separated markings.

A Breadth of Elements for Sustainable Construction

Originally forecast to accommodate 6,000 trips per day, the completed bridge is experiencing up to 10,000 trips per day and will reduce car crossings by up to 84,000 each year – lowering carbon emissions, pollution and congestion. The bridge will cut travel times by up to 50% for some users and save pedestrians up to 30 minutes when walking from Kangaroo Point and major attractions on the south side of the river, such as the Gabba stadium.

In addition to community and sustainability considerations, the Kangaroo Point Bridge features a wide range of sustainable design elements, including:

- Seventy-five solar panels on the canopy structure that can generate enough power to offset the energy usage of the bridge, including all the bridge's lighting and the lift at Kangaroo Point. The solar panels also connect to the mains supply to allow excess energy generated to feed back into the grid.
- Rain gardens at CT White Park at Kangaroo Point, to slow stormwater run-off and reduce the need for irrigation.
- Extensive use of cooling, native plants in new landscaping features on the bridge and at landings, to increase community comfort, decrease urban heat and improve biodiversity.

Other notable sustainability achievements include:

- Reduction of construction-related carbon emissions by 19% from the reference design.
- Procurement of local steel, reducing emissions by 3,339.4 tCO2e, accounting for 16% of the total construction emissions reduction.
- 59% reduction of total project emission footprint (construction and operation) compared to the base case (preventing 34,543 tCO2e).

Traditional custodian engagement informed the design, driving the visual intent and contextual grounding in First Nations knowledge-sharing. Site-specific cultural stories were individually positioned to strengthen cultural connection at points along the bridge. For example, history of the traditional Bunya Festival, fishing practices and skills of Indigenous people are captured in plaques on and around the bridge.

Detailed Design Development

The 460m overall bridge length has a 183m main span and a 90m asymmetrical back span, plus approach spans. With a clean superstructure line and single sculptured mast, the cable-stayed main span limits the number of piers in the Brisbane River, creating a visually and technically elegant solution with a light footprint while also providing benefits for flood performance and clearance for the robust Brisbane River ferry system.

Foundation Design

The Kangaroo Point Bridge alignment traverses a range of geological formations and conditions. River bedrock formations are the Neranleigh Fernvale beds which consist of sedimentary rocks overlaid with river alluvium.

The abutment and pier piles within the river including those supporting the main span are founded in these Neranleigh Fernvale sedimentary rocks.

← Figure 3: The bridge touches down in the lush south bank of the Brisbane River

The eastern side of the river is a raised bank, which has Brisbane Tuff and an underlying unconformity transition zone – the pier and abutment foundations on this side of the river are founded in the Brisbane Tuff.

Pile group geometries were optimised to ensure that the design complied with substructure blockage requirements and limited obstruction to the waterway as required by flood modelling.

The piles for the river piers varied in size: 2.1m-diameter cast-in-place reinforced concrete piles supporting the main span (at Piers 3, 4, and 5), with 1.5m and 0.75m piles elsewhere.

Pile lengths varied with load and founding geology and were approximately 26m long at Pier 4 under the mast, with a minimum 6m sockets in the Neranleigh Fernvale (Argillite) founding material at this location.

Dominant load effects for foundation design for river piers were typically from vessel impact, with major river piers designed to resist up to a 2200 tonne displacement barge travelling at 8 knots.

The performance requirements limited tension in rock sockets under permanent loads. The design of the anchor pier at the Pier 3 back span foundation was a key design challenge due to the large asymmetry between the main and back spans which required the foundation to act as a “tie down” pier.

Figure 4: The selected alignment links the CBD and Botanic Gardens (near side of the river) with Kangaroo Point (far side of the river)

A single row of large diameter piles was the preferred design solution as it minimised permanent tension in the rock socket while still providing the necessary strength to resist vessel impact.

Additional measures included adding counterweight in the back span in the form of concrete ballast within the steel box (impact of increasing foundation mass was minimized due to buoyancy effects) and including a limiting uplift force target in stay cable force finding.

Pile Caps

The constructability of the river pile caps was improved using a precast pile cap shell and skirt permanent form arrangement.

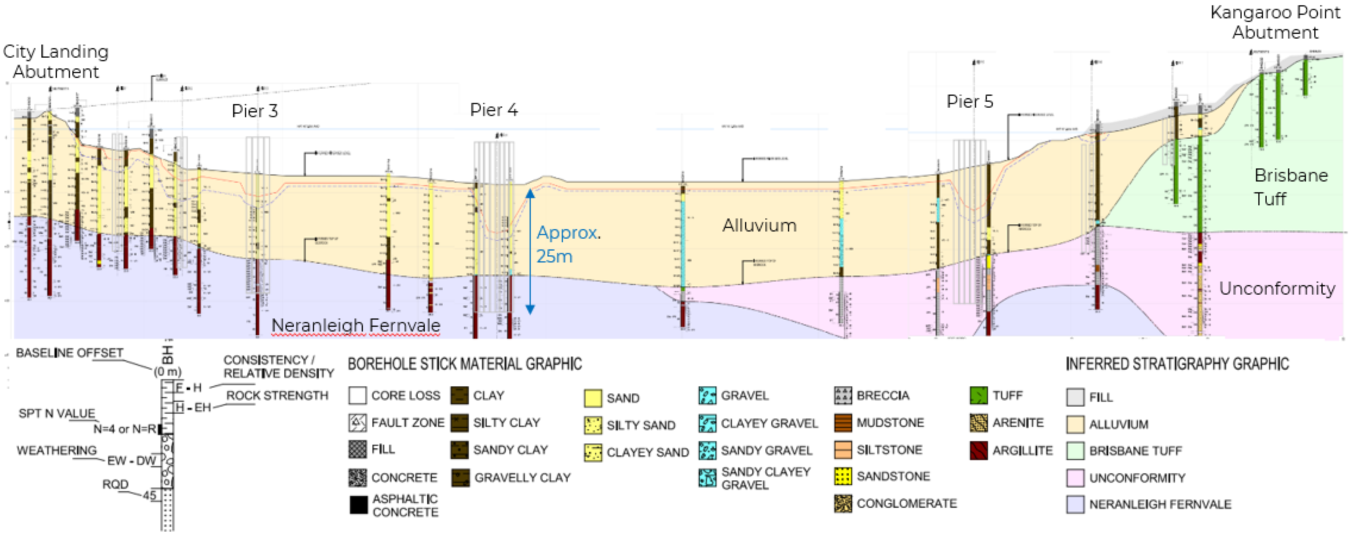


Figure 5: Geological long section

The shell was barged to each pier and positioned over the piles with the base level set above the low tide level. For the river piers with single rows of piles (Piers 1, 2, 3, 6) the pile cap shells were precast as single units. For the larger main span piers (Piers 4 and 5) comprising 8 piles in 2 rows, the precast shells were manufactured in 3 units, assembled over the pile group, sealed and dewatered (temporary support of these large shells was provided using steel plunge columns cast into the piles).

After placing the shell, the precast concrete skirts were placed around the perimeter and secured with bolted attachments. The skirts were detailed so the toe remained below water at low tide level, ensuring the piles are not exposed at low tide.

To strengthen the base of the shell, a first stage concrete pour was cast after placement of the bottom mat reinforcement. This allowed a reduction in the precast shell base and wall dimensions, with weights minimised for improved lifting and handling. The remaining depth of the pile cap was then cast on the strengthened base as a second stage pour.

Piers

The project architects developed a family of pier shapes which gave the bridge its striking sculptured form. The fork shaped piers used at Piers 1, 2, 5, 6 and 7 were developed to have consistency in the cross-sectional shapes while allowing for variations in heights and support widths at deck level.

The base dimensions provided the required structural capacity to resist loading from SLS and ULS flood events concurrent with "large item" impact loads.

Pier 3 provided the back span "tie down" and is a solid pier with a faceted arrangement. Tie down was achieved using plain column reinforcement extending from the pile cap to the deck diaphragm. This was preferred over post-tensioned bars due to the difficulty in maintaining compression under permanent loads at the deck/pier interface.

Pier 4 supports the main mast tower and comprises a "cradle" form that transitions the four legs of the upper mast arrangement to a single 4.6m diameter column. The upper legs of the cradle are laterally tied at deck diaphragm level.

The complex detailing arrangement catered for factors including the staged construction requirements, complex geometry at the mast leg interfaces, and complex loading from the mast legs, requiring anchorage of significant diaphragm tie bars to resist horizontal thrust forces and deck load reactions from cable-stayed spans.

The diaphragm also has embedded steelwork from the cantilevered composite deck for the "pause point" observation platforms.

An internal void within the diaphragm was required to house and access the bridge utilities that run end to end of the bridge.

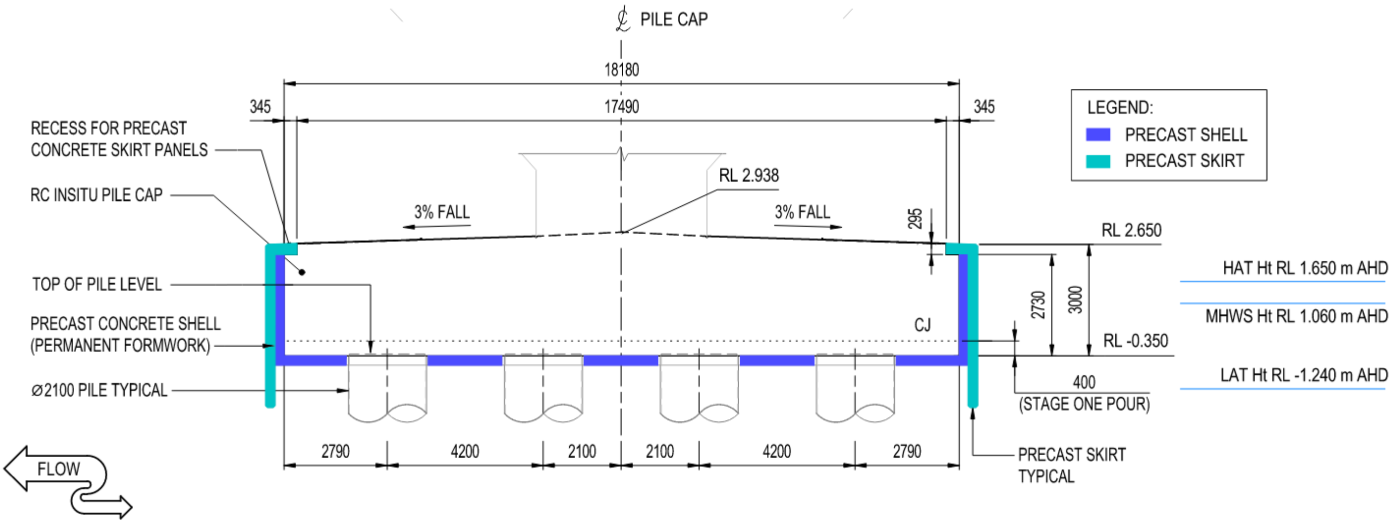


Figure 6: Typical pile cap arrangement (river piers)

Figure 7: Typical forked pier arrangement; Architect's Rendering (left), Structural Digital Model (right)

The reinforced concrete diaphragm at Pier 4 was cast in multiple stages: the first stage supported the steel outriggers prior to being cast compositely with the second and third stage diaphragm pours.

Main Span Superstructure Design

The main span superstructure comprises a constant width, 1.2m-deep steel box girder with a composite concrete deck and internal bracing. Bespoke, partial-depth precast panels were designed and used to function as permanent formwork for the in-situ deck concrete pour.

Girder sections also incorporated a continuous steel edge beam on both sides of the deck, connected with regularly spaced outriggers.

For the typical section of the bridge, the outriggers support the deck cantilever as well as provide the desired architectural form. At cable-stay anchorage locations, outriggers are designed to transfer loads back to the steel composite box girder.

A Striking Mast Design

The mid-river mast structure is the bridge's landmark feature. The fabricated box steel structure anchors the stay cables supporting the main span, with a crown at the top of the mast approximately 95m above the river's highest astronomical tide level.

To enhance structural performance and constructability of the four mast legs, a mid-height cruciform cross bracing member was introduced.

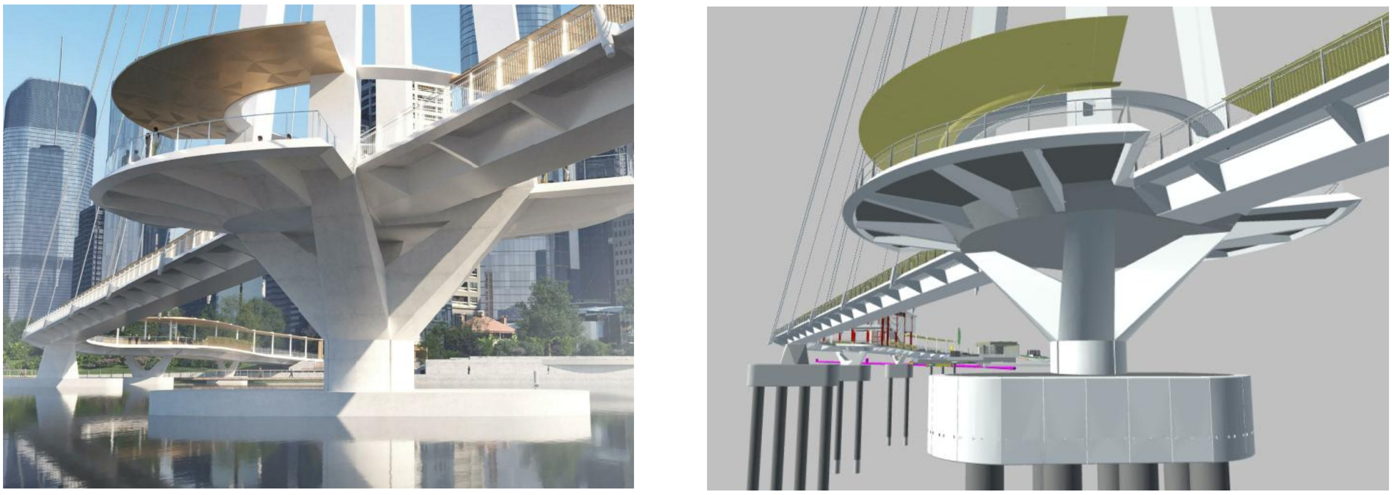


Figure 8: River Pier (Pier 4) arrangement; Architect's Rendering (left), Structural Digital Model (right)

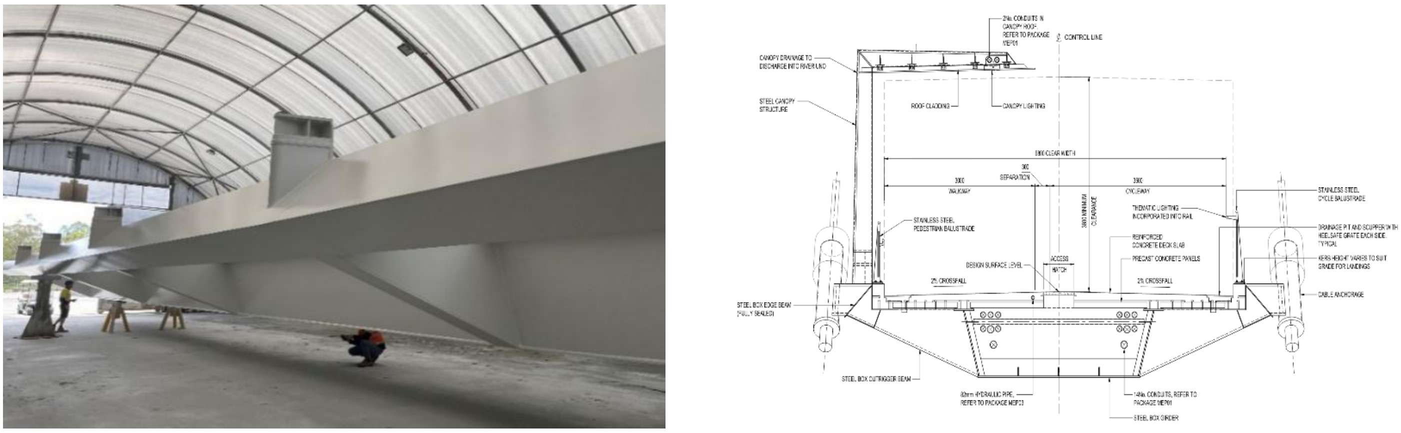


Figure 9: The steel box superstructure shape resulted in a slim. clean design while providing designated pathways and separation between transport modes

The cruciform improved the performance of the mast legs under axial load, reducing structural dimensions and saving on steelwork, thereby reducing project cost.

The mast leg form is a scalene quadrilateral, tapering over the mast height to achieve the desired appearance, with side lengths ranging from 1.7m to 2.1m. The legs are oriented diagonally relative to the longitudinal/transverse axis of the bridge to improve wind performance and to tie geometrically into the plane of the cradle legs.

The mast head is accessible internally for inspection and maintenance of the stay cable anchorages and mast feature lighting. Access doors are concealed at the base (above a concrete plug) and internal ladder access is provided in one of the mast legs. The mast head structure includes significant steelwork to anchor the stay cables. Fabricated in a single piece and lifted into place, it weighed approximately 180 tonnes.

The mast steelwork was designed for loads established from global staged analysis models. These load effects and local stress concentrations at connection details were further studied using finite element models. Introduction of the mid-height cruciform provided sufficient restraint that the mast legs avoid global buckling prior to forming plastic hinges. The mast design was governed by section capacities with considerations of local buckling of the longitudinally stiffened leg panels.

Mast horizontal deflection at the end of construction was determined using the proposed cable-stressing forces and construction sequence.

The final horizontal deflection is moderate at <50mm for the 95m-high mast structure (span/1800); therefore, no pre-camber to the mast steelwork was necessary.

Stay Cables

Stay cables were specified as a parallel strand system allowing for individual strand by strand replacement. Typical cables are either 12 or 15 strands, with the exception of the twin pair of backstay cables anchored at Pier 3, which are 43 strand cables due to the larger backstay anchor forces.

The cable anchorages were specified as proprietary type with a protection system for long term durability complying with the Fib Bulletin 89. Cable pipes were specified with double helical fillets for improved wind-rain vibration performance and with vandalism protection pipes at deck level. Fatigue testing requirements were also specified, in line with the Fib Bulletin 893, with a maximum SLS load target in the cable set at 50% ultimate tensile strength. Cable loss and replacement combinations were also considered.

The cable anchorages are designed to resist the Fib Bulletin 89 requirement of 90% of the cable ultimate tensile strength. This overload is applied to individual cables and is a ductility provision to ensure overstrength of the anchorage allowing post yield deformation of the cable. At the top of the mast anchorages, each cable anchorage supports two cable pairs (one for each side of the deck). The cable anchorage consists of two parallel transfer plates that are designed to transfer the anchorage loads to the side walls of the mast core steelwork.

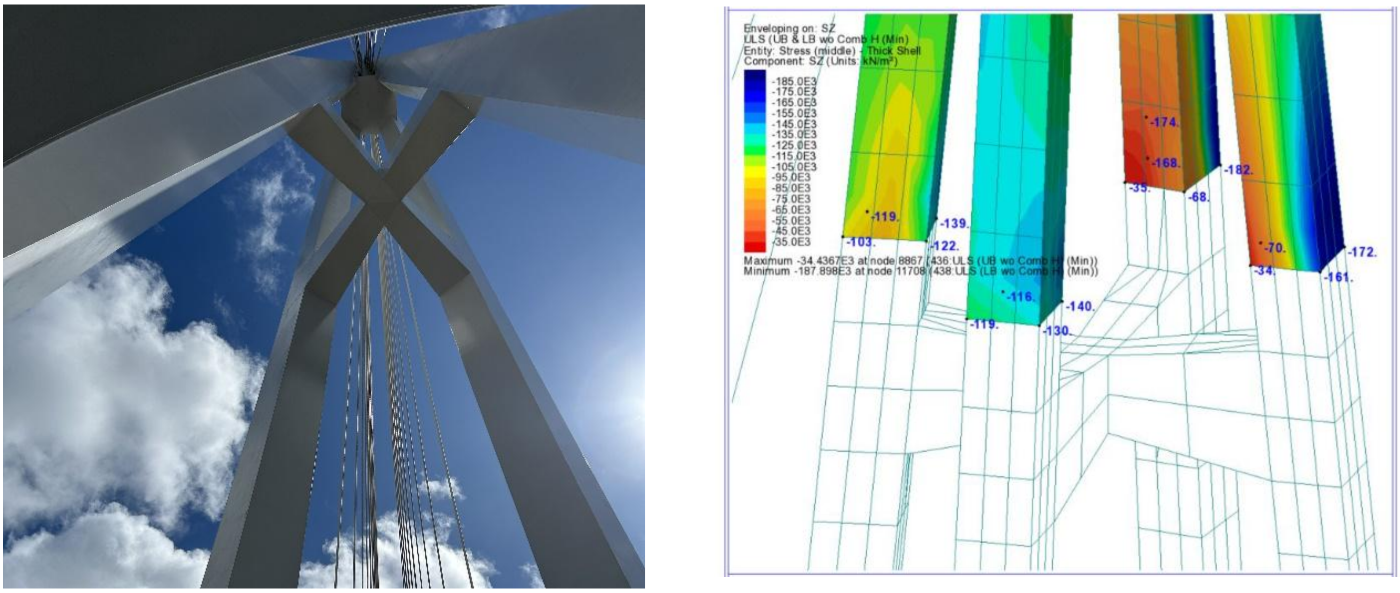


Figure 10: The steel box legs of the mast are kept slender with the help of a cruciform bracing part way up

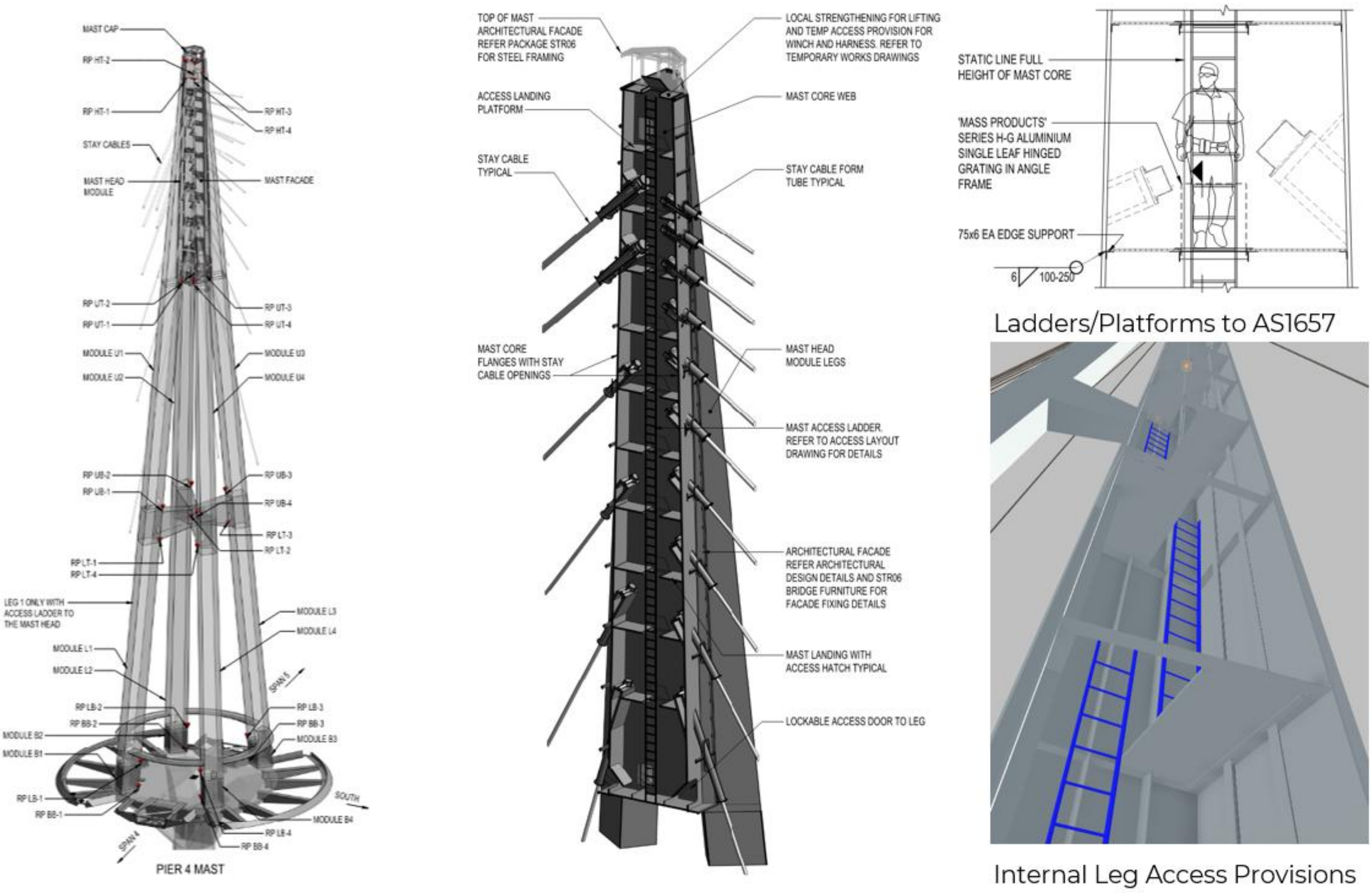


Figure 11: Mast arrangement and maintenance access

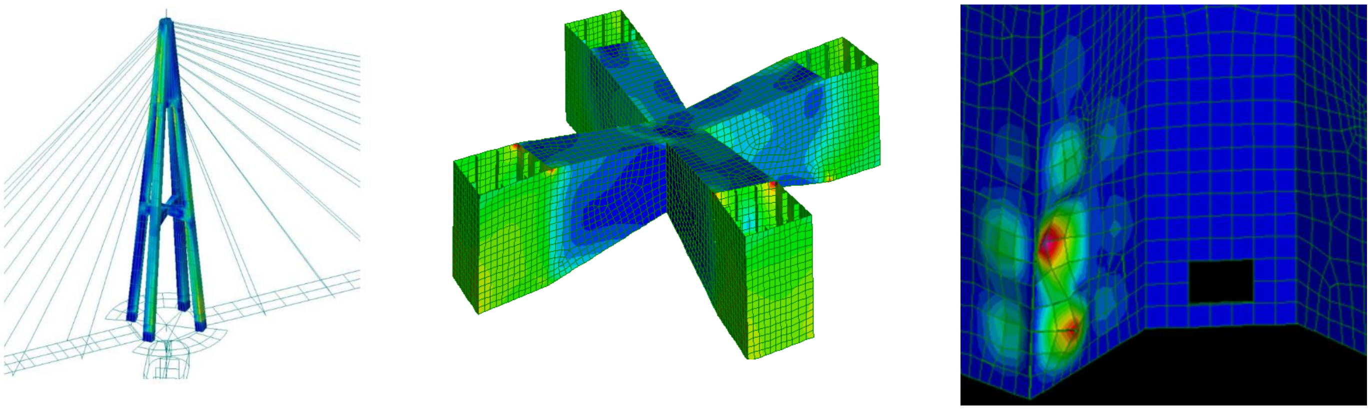


Figure 12: Mast design analysis examples: Global Model (left), Local Finite Element Model (Middle), Local Plate Buckling Analysis (right)

Each anchorage has a separate bearing plate and associated sealing plates to form a box section for load transfer. The stay form tube is welded into the anchorage to provide durability protection, and the anchorage plates are also internally sealed to avoid the need for future maintenance of steelwork protection coatings.

Due to space constraints, the depth of the transfer plates was made as shallow as possible. The largest plate is 80mm thick for the larger main backstay cable anchorage, and 60mm thick for the typical stay cables.

At deck level, the anchorage is formed using a heavy gauge form tube which combines the functions of transferring the stay cable anchor forces to the edge beam, providing the durability protection to the stay system, and housing any cable vibration damping systems where required.

The cable damper consisted of an internal viscous piston damper attached to the form tube and clamped to the cable.

The design incorporates a housing that is free to move with the cable, providing the required durability, vandalism protection, and access for maintenance of the damper system. The design was coordinated to provide the required architectural finish while expressing the necessary design functionality – visualisations of the design were presented for approval.

Stay cable anchorages are connected to the deck using fin plates welded to the girder edge beams. Steel outriggers at stay supports also connect the stays back to the deck box at internal diaphragm locations. The loads are inclined and vary from one stay to the next to suit the changing form pipe angles and varying loads.

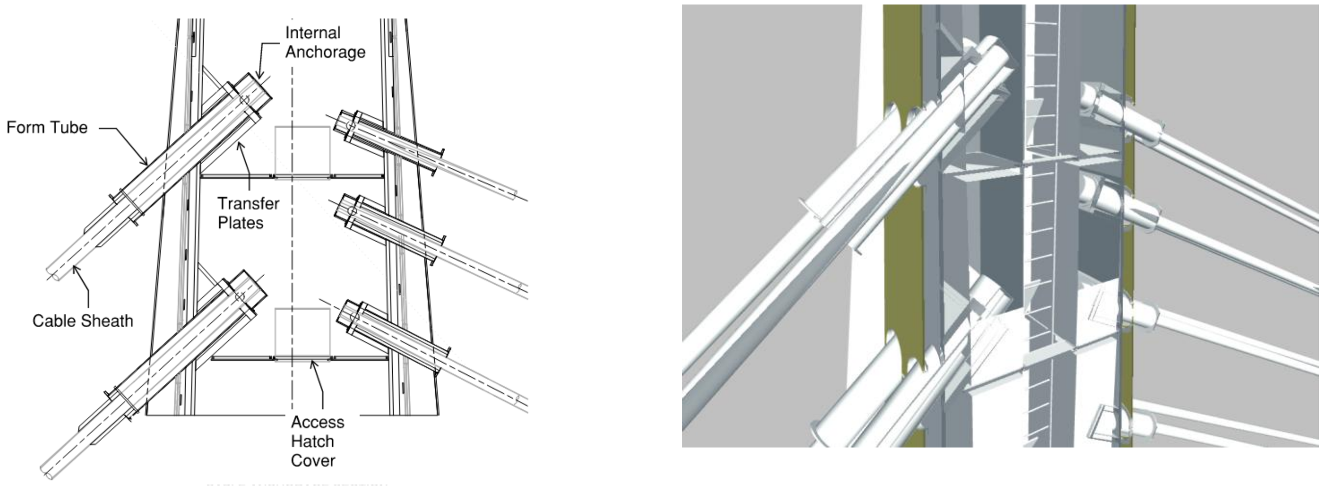
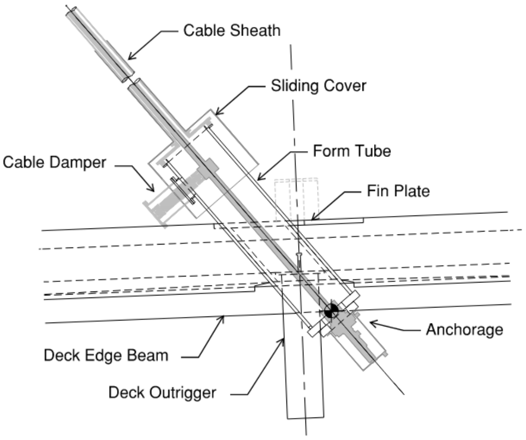


Figure 13: Pylon mast stay cable anchorages; Section through mast head at stay cable anchorages (left), Detailed Design Digital Structural Model (right)



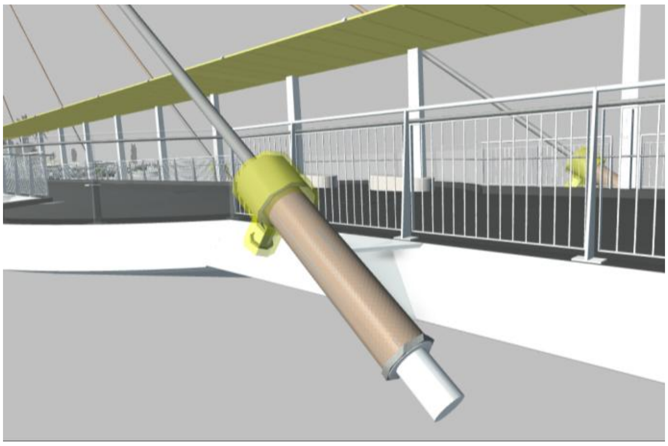


Figure 14: Deck stay cable anchorages; Stay Cable with Integrated Cable Damping System (left); Detailed Design Digital Structural Model (right)

The anchorage forces are transmitted into stresses in the deck steelwork and composite concrete deck, with shear studs provided along the length of the edge beam.

Construction Stage Analysis

During the detailed design phase, construction-stage analysis was conducted with the contractor's erection engineer to refine stay-cable sizes and tensioning loads, achieving optimal deck levels and design.

The construction method was integral to the deck design, influenced by the innovative scheme of erecting long, prefabricated modules on temporary pier supports before forming welded splices, casting the composite deck, assembling stay cables in a defined sequence, and stressing them in multiple stages.

Construction-stage analysis was performed using SOFiSTiK and validated using MIDAS Civil software. Non-linear effects such as second-order effects, cable sag and time-dependent effects were considered. Analysis was complicated by the temporary portal supports used to erect the main span modules, which needed to be activated and deactivated at a determined point of “lift off”.

In some cases, the supports were still under load after stressing and needed to be lowered using jacks to disengage the support. This novel and complex staging of cable stressing achieved precise tolerances, threading the deck profile through a narrow space for vertical clearance and gradient limitations. It balanced forces between the back and main span, allowing for significant reduction of a concrete fill counterweight. This resulted in significant cost and carbon savings for the project.

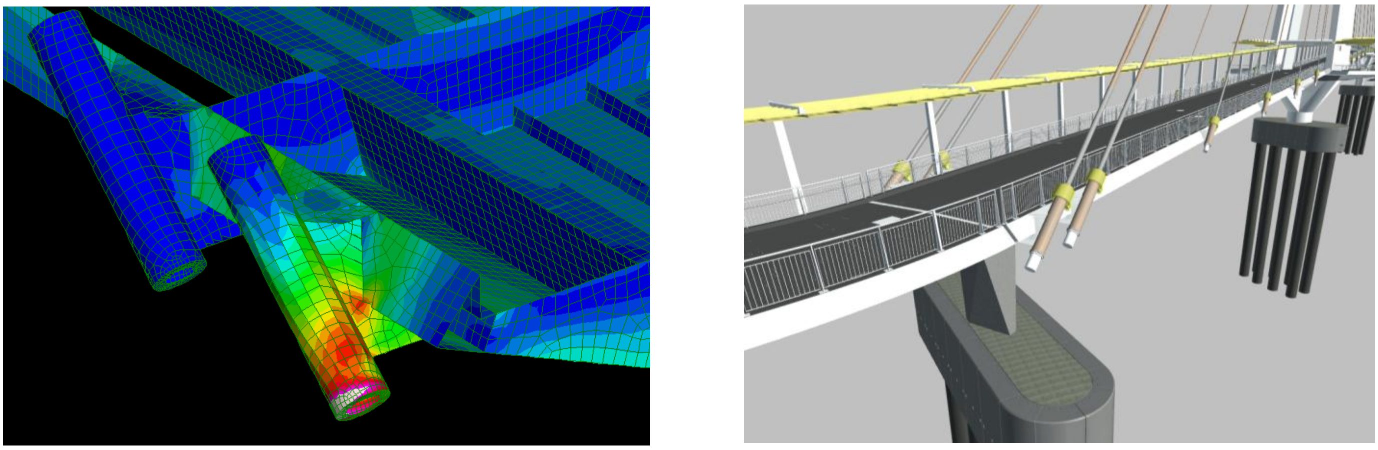


Figure 15: Backstay cable anchorages; FE Analysis of Back Stay Anchors (Concrete Not Shown) (left), Detailed Design Digital Structural Model (right)

Cable Force Finding and Pre-camber

Cable force finding and deck level optimisation were performed in SOFiSTiK using the construction stage module together with an iterative solver that determined the required stressing forces to achieve nominated force targets in the deck and mast, as well as other constraints. For example, other constraints considered in the analysis included maintaining permanent compression in bridge bearings at Pier 5 and 6.

A force target approach was used in lieu of a displacement target approach as it typically leads to a more optimised solution in terms of force demand in the deck and mast. With this method, the design requires moderate cambering to achieve the design profile of the structure. To incorporate the second order effects resulting from pre-camber, the model is automatically updated with the pre-camber geometry.

Cable forces are checked against the SLS and ULS limit, which was set at 50% and 74% of the guaranteed ultimate tensile strength (GUTS) in accordance with Fib Bulletin 89. During construction, the SLS limit was increased to 60% of GUTS upon first stressing in line with the provisions of Eurocode EN 1993-1-11.

Once the target forces were confirmed, the deck pre-camber was determined by specifying a “zero deflection” target at the completion of the bridge when the design geometry is to be achieved. The calculated pre-camber accounted for the modules being spliced tangentially with the previous module to achieve a perpendicular design splice orientation.

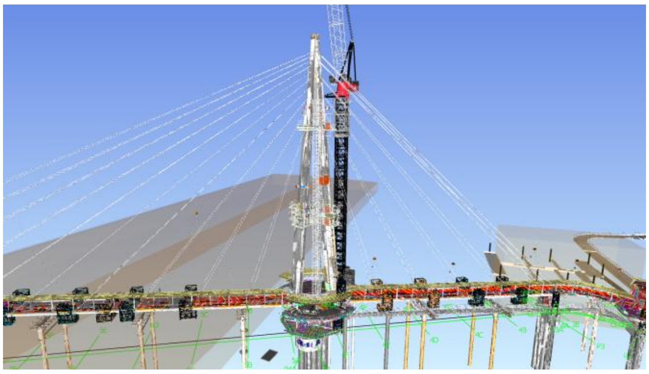


Figure 16: Complex geometry and integration of structural elements with services on the bridge was made possible through the use of LOD 500 modelling which integrated design and construction needs

Vertical adjustment of temporary works portal levels was required to account for this method of erection. Each main span module pre-camber in the erected position on temporary works was determined, with pre-camber and level sets during construction documented on the drawings.

Deflection control was refined as part of the construction engineering phase, while the stay stressing load and sequence and deck module pre-camber were specified during the design phase. This placed constraints on the construction.

Deck modules were weld spliced together at particular construction stages, which varied in level by up to 500mm at the time of splicing from the final vertical design position. The erected deck modules needed to follow the target profile to achieve the final levels, with segments spliced at each stage to suit the target profile.

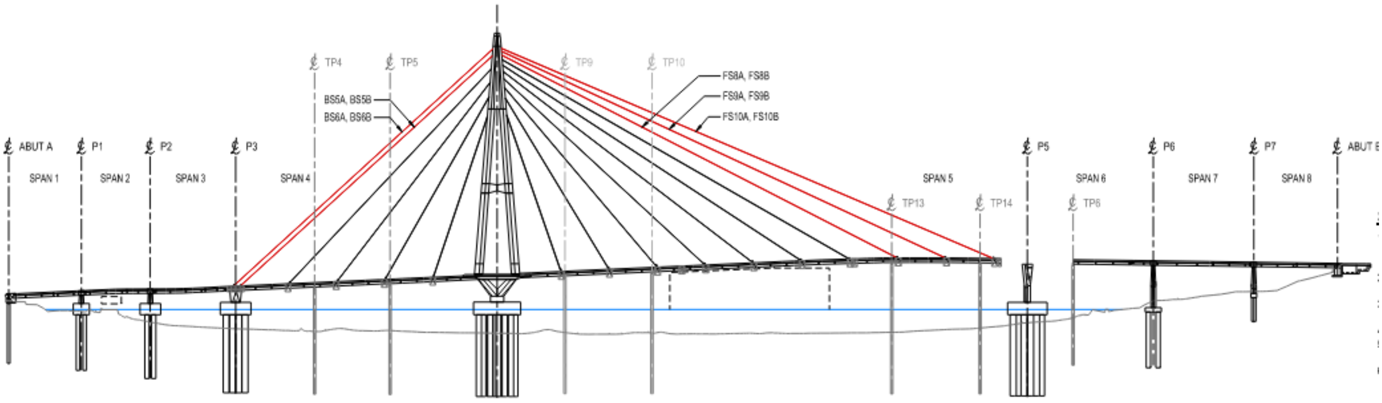


Figure 17: Construction staging across the river accommodates both construction activities and regular maritime and ferry traffic without interruption

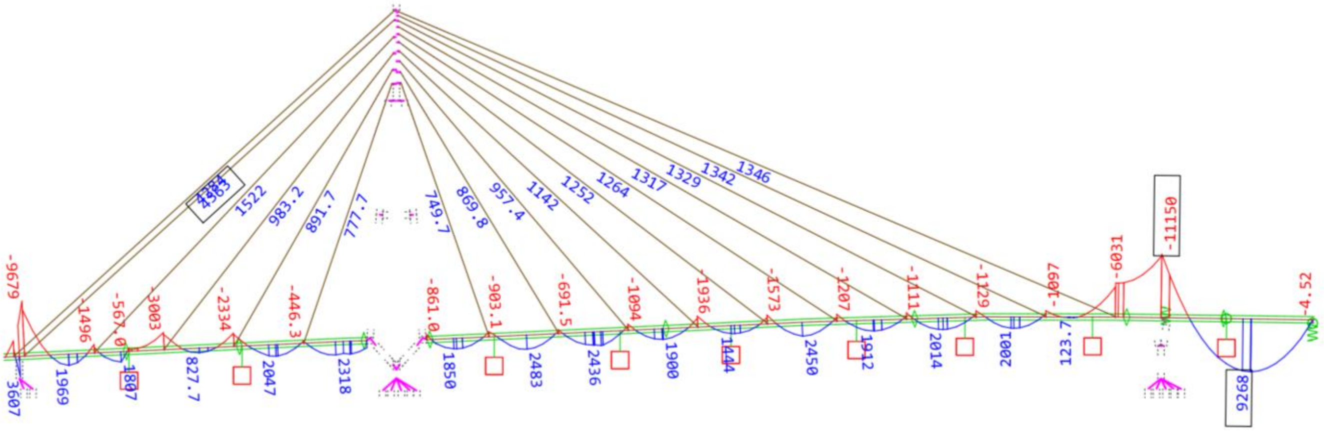


Figure 18: Optimised permanent effects bending moments and cable forces at completion

Wind and Pedestrian Vibration Analysis

Criteria for pedestrian comfort relating to the bridge's dynamic response under wind and pedestrian footfall were evaluated through analysis and in-situ testing, resulting in provisions for a tuned mass damping device.

The bridge has a long span relative to the width of the deck. As a "wind sensitive" structure, detailed wind studies were undertaken progressively to validate the design. The wind study was developed collaboratively by the structural designer (WSP) and wind consultant (RWDI), and comprised the following components:

- Wind Study Specification (by Structural Designer). This was developed early in the design phase to define the wind study requirements, including the outputs needed to inform the structural analysis. The specification also provided the necessary design inputs from structural design (structural model frequencies, section properties etc.)
- Wind Climate Analysis (by Wind Consultant). This documented the site-specific design wind speeds and turbulence properties, considering wind conditions at the site, historical meteorological data and local topography
- Desktop Aerodynamic Stability Assessment (by Wind Consultant). Design review of aerodynamic stability by empirical calculations, and interpolation from previous wind tunnel tests of similar bridges. This provided the Design Team with early indications of the stability of the structure and is used to determine whether further tests should be undertaken.
- Wind Tunnel Test 1 – Free Standing Pylon Model (by Wind Consultant). Force-Balance Wind Tunnel Test on the pylon (mast) to determine the overall aerodynamic characteristics. Used for the definition of pylon force coefficients.
- Wind Tunnel Test 2 – Deck Sectional Models (by Wind Consultant). Wind tunnel test to determine the aerodynamic stability of the deck (vortex induced oscillations, galloping, flutter), deck drag and lift coefficients, as well as aerodynamic derivatives used for buffeting assessment. Tests were completed with and without the canopy. No instabilities were observed.
- Wind Tunnel Test 3 – Aeroelastic Model (by Wind Consultant). Comprehensive approach for determining the aerodynamic stability of the bridge. Accounts for the 3-dimensional effects of the bridge that cannot be captured by other testing methods and includes the surrounding terrain and buildings.

  Completed bridge and critical construction stages studies were considered. Measured responses and loads were used to validate results from the Structural Designer's wind buffeting analysis. The test concluded that the deck and mast are expected to be stable against aerodynamic instabilities.
- Numerical Wind Buffeting Analysis (by Structural Designer).

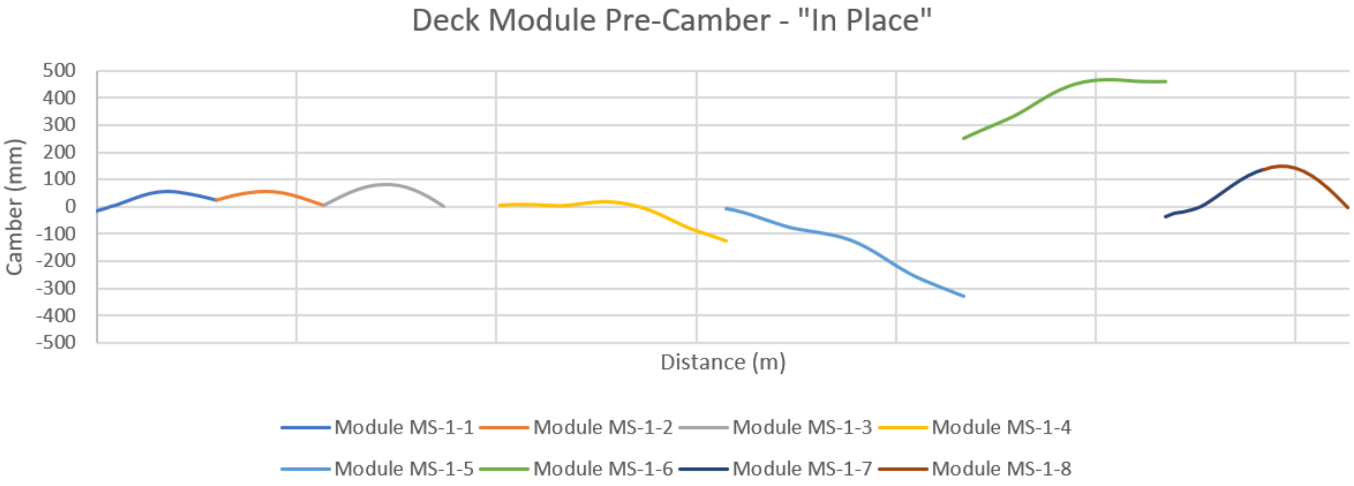


Figure 19: Deck module pre-camber in as erected position

Numerical Wind Buffeting Analysis

The numerical wind buffeting analysis was performed for the following purposes:

- Correlating the aeroelastic wind tunnel test results using the aerodynamic force coefficients and wind flutter derivatives determined with the deck sectional wind tunnel test.
- Determining displacements, accelerations, and force effects, including any key design cases where not recorded and reported with the wind aeroelastic testing procedures.
- For the study of wind effects in design and construction stages, not performed in the wind tunnel testing.

The finished bridge design cases and various construction stages were analysed. Wind tunnel testing measurements were then used to validate the numerical analysis results.

Additional construction stages not covered in the wind tunnel testing were studied using numerical analysis.

The wind parameters provided by the wind consultant served as inputs for the wind buffeting analysis. These parameters included the wind profile, turbulence properties, force coefficients and flutter derivatives.

Drag forces, vertical lift forces, and deck moments per unit length were derived from the force coefficients.

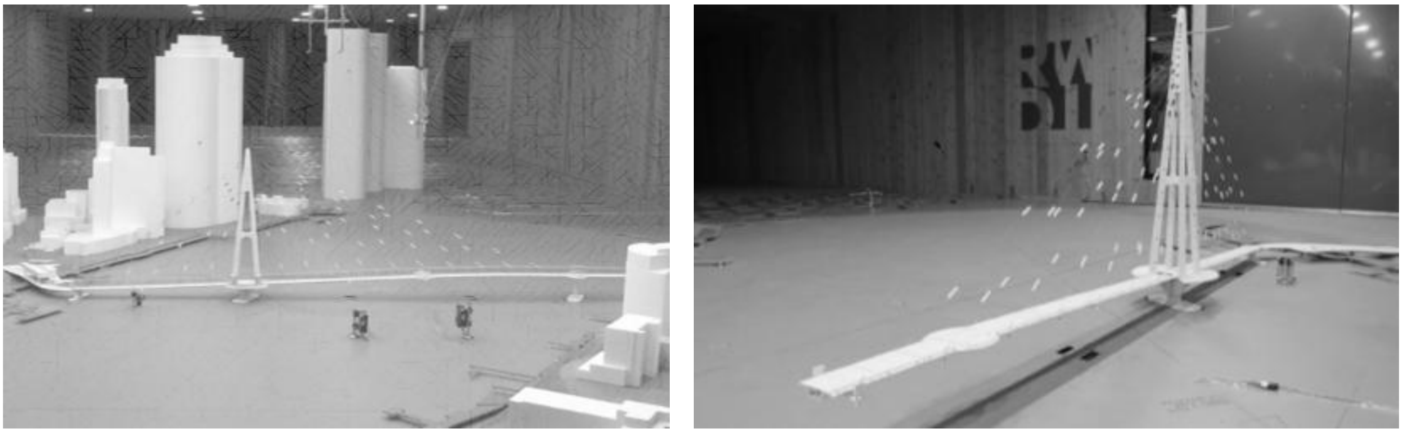


Figure 20: Critical to reliable user comfort is the wind and pedestrian vibration analysis, backed by physical testing under both the completed configuration (left) and during construction (right)

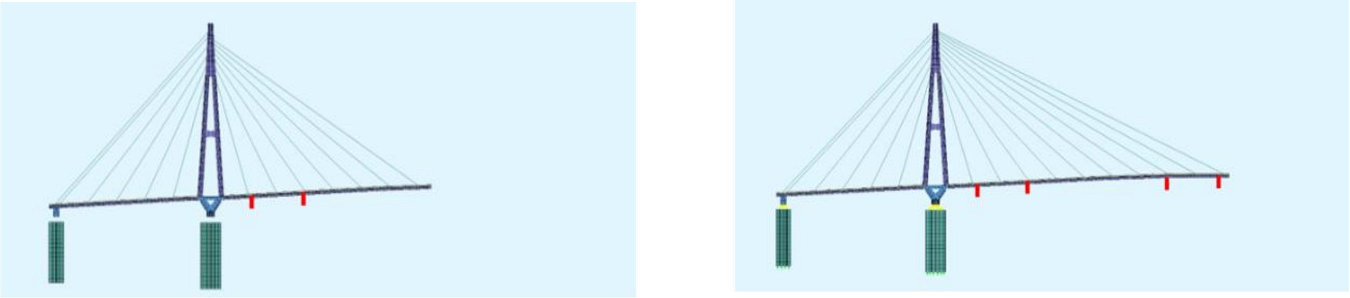


Figure 21: Wind buffeting model – cantilever stages; Main span second module cantilever stage (Stage 40) (right), Main span third (final) module cantilever stage (Stage 41) (left)

Separate force coefficients for each mast leg, determined through Force-Balance Wind Tunnel Tests for the pylon, were used, accounting for the shielding effect of wind-facing legs.

Two analysis methods were employed to simulate the dynamic wind:

Spectral Analysis (frequency domain) which is an efficient method for predicting responses with minimal processing time. However, it has limitations, including the inability to account for structural non-linearities, the requirement for pre-determined aeroelastic damping to be added to structural damping, and the potential to neglect local effects in lower eigenforms.

Transient time history analysis (time domain) where artificial wind histories were generated and analysed, considering aerodynamic damping and galloping effects.

The analysis was based on unsteady theory considering the flutter derivatives. To ensure proper aerodynamic interaction, 10-minute runs comprising 12,000 steps (0.05 seconds per step) were performed.

For statistically robust results, multiple time history runs were conducted, and the mean of the maximum values was adopted as the maximum response.

A routine was developed to automate successive time history analyses for each investigated condition. Envelope plots of the displacement, acceleration, and force histories were generated for both the spectral and time history analysis which were then used to further inform the design of the bridge. A reasonably close correlation was found between the aeroelastic testing and the transient buffeting analysis for the deck vertical response and the mast longitudinal response.

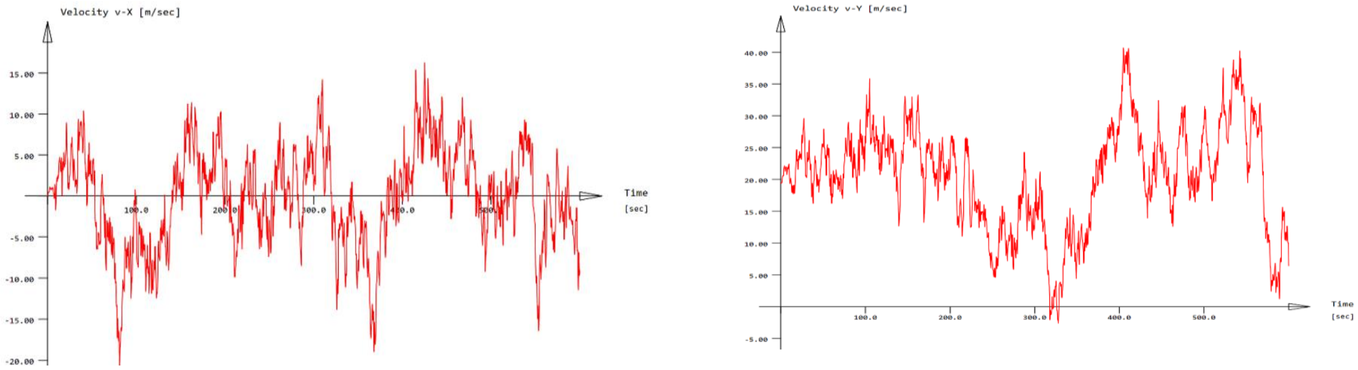


Figure 22: Typical wind histories; Transverse to wind direction (left), Longitudinal to wind direction (right)

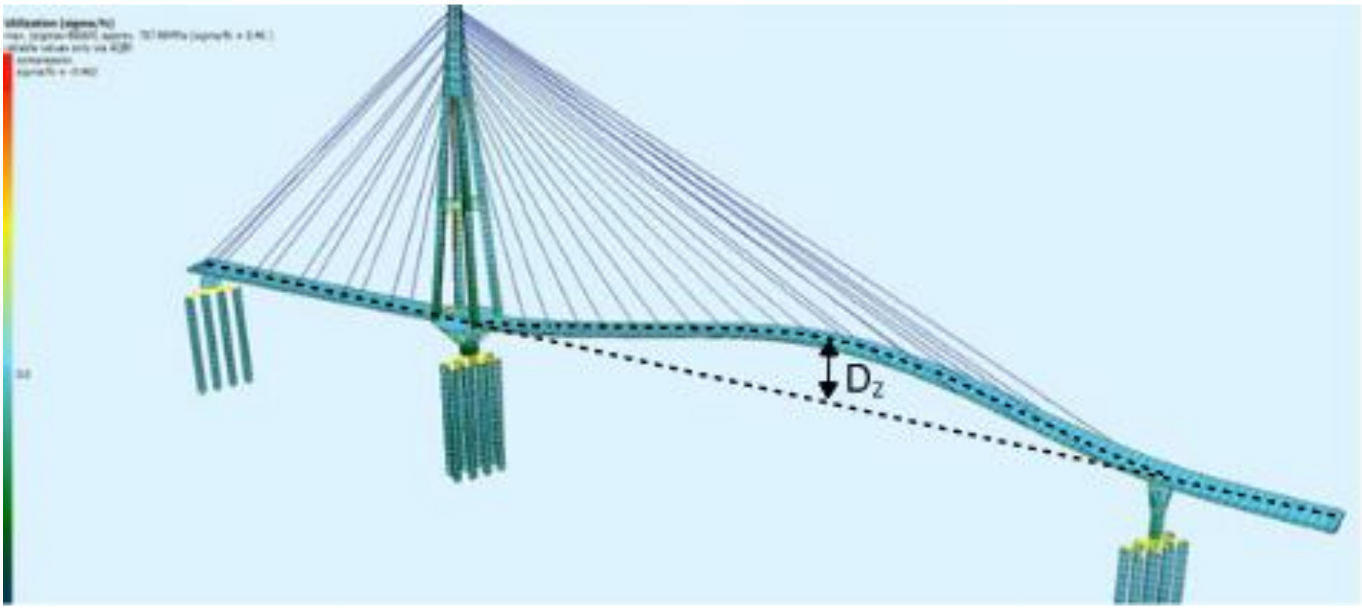


Figure 23: Numerical wind buffeting analysis – vertical deck response

Table 1 illustrates the vertical deck displacements findings for key stages. Positive values are upwards and negative values downwards.

Numerical wind buffeting analysis did provide a higher response of the mast in the transverse direction. To ensure safe design, the higher response indicated by the numerical buffeting analysis was adopted.

In addition to ULS and SLS force effects, deck accelerations under a mean wind of 15m/s were also evaluated for pedestrian comfort.

This lower speed was adopted as an upper bound before bridge closure to pedestrians due to high wind conditions.

The calculated mean peak accelerations were well within the limits for pedestrian comfort; therefore, it was concluded that no additional wind damping devices were necessary.

<u>Footfall analysis and damping requirements</u>

Dynamic assessment procedures and criteria were specified in the Performance Specification. Refer below for a summary of the cases to be considered:

- Single person walking on the bridge – The Concrete Centre Publication CCIP-016 or SCI Publication P354 (vertical response).
- Crowd loading – SETRA Technical Guide: Footbridges – Assessment of vibrational

Position	Effect	Max/ min	Completed Bridge			Construction Stage 40			Construction Stage 41		
			WT (2)	Trans. (2)	Spect. (2)	WT (2)	Trans. (2)	Spect. (2)	WT (2)	Trans. (2)	Spect. (2)
Main Span (1)	D_z (mm)	Max	140	107	123	700	452	548	-	611	965
		Min	-140	-104	-123	-670	-582	-548	-	-869	-965

Table 1: Comparison between wind tunnel testing and numerical wind buffeting analysis for vertical deck displacements (mm)

Notes:

(1) Span centre for completed bridge, tip of cantilever for Construction Stage 40 & 41.
(2) WT – Wind Tunnel Aeroelastic Test, Trans. – Numerical Transient, Spect. – Numerical Spectral.

behaviour of footbridges under pedestrian loading (pedestrian comfort vertical and horizontal response).

- Crowd Loading (no lateral lock-in) – The London Millennium Footbridge, Dallard et al (lateral stability).

The single person load model defined by CCIP-016 or SCI Publication P354 was found to result in a response factor well below the nominated limit. This is to be expected considering that a structure of this size and weight will require substantial energy to be excited.

The SETRA defined frequency ranges where there is a risk of resonance from crowd loading are 1.0Hz to 2.6Hz for vertical and longitudinal vibrations and 0.3Hz to 1.3Hz for lateral vibrations. Within this range, the maximum risk for resonance occurs between 1.7Hz – 2.1Hz and 0.5Hz – 1.1Hz respectively. Outside these higher risk ranges a reduction factor was considered.

The load model in SETRA was based on idealising a stream of random pedestrians with an equivalent stream of perfectly synchronised pedestrians.

The derived uniform load was applied harmonically according to the relevant mode shapes at a frequency close to the mode shape where the largest response was obtained.

From these studies, it was found that two lateral modes and seven vertical/torsional modes fell within the ranges that required further consideration. Dynamic analysis of these modes confirmed that the calculated accelerations were within comfort limits; therefore, no additional damping was required for pedestrian comfort.

During the early stages of design, the inclusion of a tuned mass damper (TMD) was identified to reduce the risk of lateral lock from crowd loading, based on the method by Dallard et al.10. It was noted, however, that the expression and k coefficient are specific to the Millennium Bridge.

To further investigate during detailed design, the guidance provided in SETRA Technical Guide: Footbridges was used. This Publication noted testing results from the Passerelle Solferino Bridge in France indicated that a 0.10m/s2 acceleration threshold should not be exceeded for lateral lock-in. It was found that a tuned mass damper with a damping ratio of approximately 2.5% tuned to the critical horizontal mode would reduce lateral accelerations well below the above limit, including for large crowd loading.

Pedestrian dynamics were considered for single-person walking, crowd loading, and lateral stability. The single-person load model showed a response factor well below the limit, while dynamic analysis of lateral and vertical/torsional modes confirmed accelerations within comfort limits, negating the need for additional damping.

A tuned mass damper was identified early in the design to reduce lateral lock-in risk from crowd loading. It was positioned within the box girder void in the main span, with access through a maintenance hatch in the deck surface.

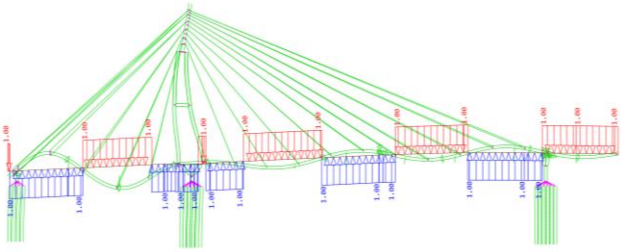
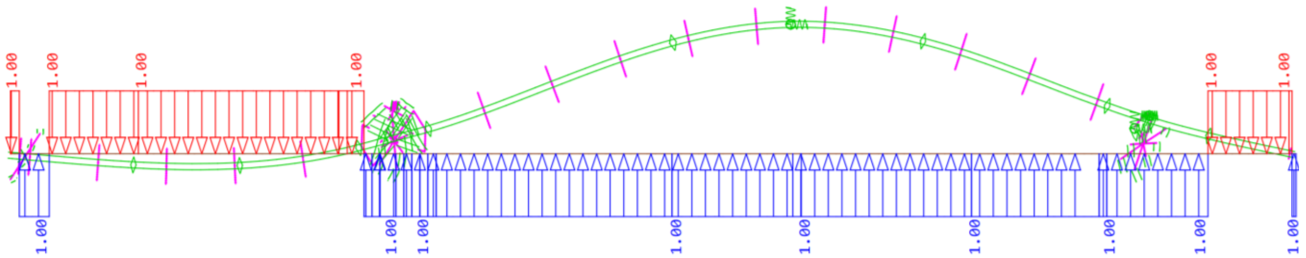


Figure 24: Example harmonic crowd loading model

↑ Key Vertical Mode (Elevation)

↓ Key Horizontal Mode (Plan View)



Cable vibration study

The risk of cable vibration is an important design and cable supply consideration. Severe vibrations lead to potential fatigue issues in stay cable system, and lesser vibrations can also lead to issues of public concern.

There are several cable vibration mechanisms, with the most common being wind vortex shedding, galloping, parametric excitations, and rain-wind induced vibrations. Cables can oscillate through excitation by the movement of cable anchorages induced by mast and/or deck motion.

Often, small movements can result in significant cable displacements if not properly considered in design. While most of the cable vibration mechanisms are dealt with by the stay cable system supplier, both parametric excitation and indirect excitation (also referred to as “external” or “cable resonance” excitation) were studied by the design dynamic analysis models.

The dynamic analysis of the cables for parametric and indirect excitation studies was separated from the overall global dynamic wind buffeting analysis of the bridge to limit processing time. The approximate numerical solutions presented in the SETRA CIP Recommendation on Cables Stays (2002) were used, which consider the coupling phenomena between the structure and the stay system through excitation of the anchorage movements.

Allowable cable vibration amplitudes were set based on the user comfort criteria guidance in US Federal Highway Administration Wind Induced Vibration of Stay Cables (Ref: FHWA-HRT-05-084) and Eurocode EN 1993-1-11. The results of the studies for these mechanisms are as follows:

Parametric Excitation: Parametric excitation occurs when the anchor excitation is along the cable axial direction and results in divergent amplitude cable vibrations. It typically occurs when the structural modal frequencies (and therefore excitation frequencies) are close to equal or twice that of the fundamental natural frequency of the cable.

Theoretically, there is an infinite number of zones of instability; however, SETRA notes that the first two zones are typically most important.

The amplitude of the response is also independent of the cable stay damping and therefore cannot be controlled by additional cable damping (however, the onset threshold magnitude can be increased with additional damping). Due to uncertainties in predicting modal frequencies, a +/-20% range of bridge modal frequencies was considered in line with Eurocode EN 1993-1-11 to determine which cables could be at risk of parametric excitation. From the numerical buffeting analysis results, the estimated parametric excitation thresholds were assessed not to be exceeded under nominal and serviceability limit state wind conditions.

The threshold values were exceeded under ultimate limit state wind conditions; however, parametric excitation was assessed to be unlikely considering the higher aerodynamic damping of cables under these extreme wind conditions.

Indirect Excitation: Indirect excitation also occurs when there is a close match between the modal frequency of the structure and the fundamental frequency of the cable and anchor movements are perpendicular to the cable axial direction.

Due to uncertainties in predicting modal frequencies, a variance range for resonance was considered to determine which cables could be at risk of indirect excitation.

The assessment indicated that amplitudes were acceptable under a nominal in service wind speed of 15m/s. Vibration amplitudes from buffeting wind were found to be below preferred limits. Higher vibration amplitudes from vortex induced vibrations were predicted, however these were still reasonable when compared to the guidance on cable vibration amplitude limits.

The assessment also indicated the potential for larger amplitudes for the longest cables under ultimate limit state wind conditions, but the limiting criterion in this case was strength rather than limiting cable displacements for extreme events.

To control the risk of the various other forms of vibrations, cables were fitted with internal viscous dampers at the deck anchorage level.

The cable dampers were enclosed in a custom design cap arrangement that also allowed the cable to vibrate freely. This damping system was specified such that a minimum total damping ratio of 0.5% could be achieved, ensuring the cable vibrations remained within acceptable limits.

CONSTRUCTION

The project used leading-edge innovation in construction through advanced engineering solutions.

A key innovation was the segmented main span, erected in three 60m-long sections. Construction also used the FAVCO M2480 – the world's highest capacity tower crane – to erect the 95m-tall steel mast, reducing the need to transport materials via truck and the amount of above-water construction in the river, as larger items could be assembled offsite.

The project offered strong local benefits during construction, with over 90% of labour and materials sourced from Brisbane or regional areas in Southeast Queensland. It provided entry opportunities into the construction industry by employing 33 First Nations people, 64 apprentices/trainees and 13 cadets, interns or undergraduates throughout construction of the bridge – promoting skill development, capacity building, and a more sustainable, future industry.

DISCUSSION AND CONCLUSIONS

The Kangaroo Point Bridge is a critical step in Brisbane's plan to move toward a more sustainable community. It creates a way for people to traverse the city using active modes of transport, away from roads and vehicles. It engages with the river environment and activates the areas around the bridge for improved community engagement.

Designing and building a project as complicated as Kangaroo Point Bridge, in a confined urban environment required the use of advanced techniques for both the permanent and temporary works design, and construction.

Parametric scripting, BIM models to LOD 500 and a wind analysis that closely correlated to wind tunnel results, were critical to achieving an outcome consistent with the architectural intent and reliable structural performance.

The result is a new icon that respects and complements the complex and diverse environment of the City of Brisbane and the Brisbane River to the benefit of the community.

Figure 25: Under construction for almost three years, the Kangaroo Point Bridge has quickly become an important element of vibrant Brisbane CBD. Construction started in January 2022 and was completed late in 2024, with the bridge opening on 15 December 2024

Figure 26: The lighting of Kangaroo Point Bridge adds a striking reference point along the Brisbane River at night

BOORLOO BRIDGE, PERTH

MULTIFUNCTIONAL LANDMARK BRIDGES IN AUSTRALIA

Susanne Bendsen
Dissing+Weitling

Figure 1: Visualisation of the Boorloo Bridge

ABOUT THE BRIDGE

With separate lanes for cyclists and pedestrians, the Boorloo Bridge (former Swan River Causeway Bridge) enhances safety and comfort for approximately 1,400 cyclists and 1,900 pedestrians who use the existing bridge daily. The bridge design includes lookout platforms and rest areas, offering panoramic views of the river and skyline.

Its proportions make it visually and physically slim and light. The cable-stayed design was chosen to minimise material use and environmental impact while integrating into the wider bridge ensemble alongside the heritage-listed Causeway Bridge (1952).

The bridge crosses an area sacred to the indigenous Australian people. While structural efficiency and durability remain fundamental, the project also responds to the immaterial values tied to this specific place.

In culturally significant and protected landscapes like this, infrastructure must address not only technical and environmental demands, but also local identity, history, and symbolism.

The bridge's winding alignment and expressive pylons are carefully adapted to the site and rooted in local folklore, landscape, and cultural references.

We wanted the bridges to literally lift heritage and history skyward with a symbol-laden pylon design that could foster local pride and user delight.

ENVIRONMENTAL CONSIDERATIONS

Located approximately 90 meters downstream from the existing road bridge, the bridge's placement minimises impact on flora and fauna while improving connectivity for non-motorised traffic.

Environmentally friendly materials were used to reduce the bridge's overall CO^2 footprint, and the city council placed great emphasis on protecting the local ecosystem, especially the water environment, wildlife, and vegetation around the bridge's foundations in Swan River.

An advanced water management system ensures that rainwater from the bridge is filtered and purified before it flows into the river.

Energy-saving LED lighting not only reduces energy consumption but also minimises light pollution in the area.

Traffic noise is mitigated with noise-reducing elements in the construction, and trees and bushes near the bridge have been replanted to compensate for vegetation removed during the construction process.

The environmental reports were prepared by engineering firm WSP and incorporated into both the process and bridge design.

INDIGENOUS RIGHTS AND STRUCTURAL SYMBOLISM

During the development phase, local project owners and project leaders made significant efforts to involve and collaborate with local stakeholders, including members of the Whadjuk Noongar community.

The bridge project is an example of co-creation with the local population around their cultural heritage. Previously excluded stakeholders have been engaged and heard.

The recognisable structure and pylon design reflect Whadjuk Noongar culture with clear references to prominent Whadjuk Noongar figures, including a boomerang representing the warrior Yagan and a digging stick (wanna) representing the indigenous rights activist Balbuk.

Figures 2 and 3: Visualisations of the Boorloo Bridge

Dissing+Weitling worked intensively with site-specific research, local symbolism, and cultural history throughout the design process. Through a series of iterative 3D design studies, the final pylon design evolved into a structural expression rooted in the identity and narratives of the place.

Beyond addressing an infrastructure issue, the Boorloo Bridge is a concrete example of the global demand for bridge designs that support local pride and heritage and serve as a catalyst for the international branding of a place.

Dissing+Weitling is no stranger to this trend — bridges in China and Canada also express local symbolism and support social and local values. The Perth bridge is a clear exponent of this movement towards structures with strong local anchoring and recognisable forms.

For Dissing+Weitling, bridge design always revolves around maintaining its design integrity and DNA, known for prioritising simplicity and buildability. Even when projects call for highly recognisable forms, structural clarity and buildability remain central.

The symbolism in the winding path, wanna, and boomerang pylons, the details on the bridge's interior, and the views from the river road are all integrated — never detached decoration — but are individually significant and collectively meaningful elements.

In 2023, Dissing+Weitling explored this design trend in the paper *"Structure as Symbolism: Pylons as Tools for Cultural Expression in the Asia-Pacific Region,"* IABSE 2023.

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Figure 4: Aerial View of the complete Boorloo Bridge with the Matagarup Bridge

BOORLOO BRIDGE, PERTH
AN AMBITIOUS ACTIVE TRANSPORT PROJECT

Wolfram Schwarz, WSP Australia

Risto Kiviluoma, WSP Finland

Gerhard du Plessis, Engineering Consultant, Australia

Figure 1: McCallum Park Bridge with sparkling lights

INTRODUCTION

The existing Causeway Traffic Bridges provide one of four pedestrian and cyclist crossings of the Swan River and carry peak-hour volumes exceeding 150 cyclists and 200 pedestrians. The existing shared path is approximately 1.8 m wide and includes uneven surface conditions and mixed user movements; these constraints raise safety concerns and limit capacity.

The Causeway Pedestrian and Cyclist Bridges Project provides a new active-transport route approximately 100 m downstream of the existing Causeway, extending about 1 km from Point Fraser via Heirisson Island to McCallum Park.

The shared path has a total width of 6.0 m, comprising 3.5 m for cyclists and e-scooters and 2.5 m for pedestrians. The delivered scheme comprises two cable-stayed bridges with three in-river piers, reducing in-water construction and acknowledging the cultural significance of the river corridor (Derbal Yerrigan) to First Nations peoples.

This paper documents the key structural design-development decisions and the wind, cable, and footfall dynamic assessments that informed the final configuration, together with the adopted erection sequencing.

During concept development, input from the local Indigenous community informed architectural motifs for the pylons: Point Fraser Bridge comprises a single 52 m pylon representing a boomerang, and McCallum Park Bridge comprises two 46 m pylons representing digging sticks.

For both bridges, stay cables are anchored along one side of the deck and connected to the pylons via a rigid link. Both bridges opened on 22 December 2024 and were named Boorloo Bridge, reflecting the Noongar name for Perth.

DESIGN DEVELOPMENT

Planning of the alignment

The approach alignments and tie-ins were developed to provide direct connectivity to existing pedestrian and cycling networks while maintaining constructability and minimising environmental and heritage impacts.

On the Point Fraser side, the alignment connects to the principal shared path serving the Perth CBD foreshore.

On the Victoria Park side, the alignment interfaces with the bus transfer station, the Canning Highway underpass, and the South Perth to Belmont cycling and pedestrian routes. The alignment was also constrained to reduce impacts on established vegetation and to avoid environmentally and culturally sensitive areas.

Figure 2: Location of the Bridge
Click on the image to open Google Maps

An S-shaped horizontal alignment was adopted to provide an organic plan form consistent with the island setting and to respond to architectural intent and stakeholder feedback. The alignment was also informed by local cultural narratives, including the Wagyl, which is associated with the creation of the Swan and Canning rivers and other landforms.

Form finding of bridge elements

The deck cross-section

The reference design adopted a symmetric deck cross-section. Because the horizontally curved decks are supported by stay cables on one side only, the cross-section was revised to an asymmetric arrangement.

Figure 3: Main Roads Western Australia official pocket map of the Boorloo Bridge

A torsionally stiff closed steel box transfers loads to the stay cables, pylons, and abutments. The initial refinement focused on structural efficiency and functional requirements and was subsequently adjusted in collaboration with the architects to achieve slimmer proportions and simplified lines. The cantilevers were required to appear as closed sections; this was achieved by cold-forming the outriggers to a V-shaped cross-section and eliminating longitudinal stiffeners (Figure 7).

Revision of the pylons

The reference design included tie-back cables anchoring the pylons to onshore anchor blocks. The design team assessed the vulnerability of the tie-back anchorages to vandalism and the associated consequences for structural stability, noting limited redundancy in the tie-back system. The tie-backs were therefore removed. In addition, given the soft ground conditions, the revised scheme reduced the number of permanent piles as well as vegetation clearing to allow construction of those piles. The change improved operational safety and reduced long-term maintenance requirements.

The reference design provided discrete struts to support the deck from the pylon pile caps. This arrangement was revised by integrating the support into the pylons to form a moment-resisting connection between deck and pylon.

The project brief required three west-facing pause points; these were located at the pylons. At Point Fraser Bridge and the first pylon of McCallum Park Bridge, the pause points were formed by locally widening the deck through increased steel box width from 3.0 m to 5.0 m which widened the clear deck width to 8.0 m (Figures 9 and 11).

Figure 4: Artist impression of the Wagyl

Individual bench seats were located at those pause points, allowing users to take in the views. At the second pylon of McCallum Park Bridge, the pause point was configured to wrap around the pylon and incorporated integrated seating (Figure 9).

The pylon geometries were refined in response to feedback received during the design competition and subsequent engagement with Noongar Elders. The McCallum Park Bridge pylons are tapered, with a diameter of 1.5 m at the base increasing to 1.8 m at the top.

Multiple pylon-tip configurations were developed to represent digging sticks. For the Point Fraser Bridge, multiple forms of boomerangs were presented before a preferred option was selected for each, with input from the Matagarup Elders Group, based on the best representation of historical forms (Figure 5).

Figure 5: Original digging stick and boomerang shapes

Figure 6: Preferred option – reference design

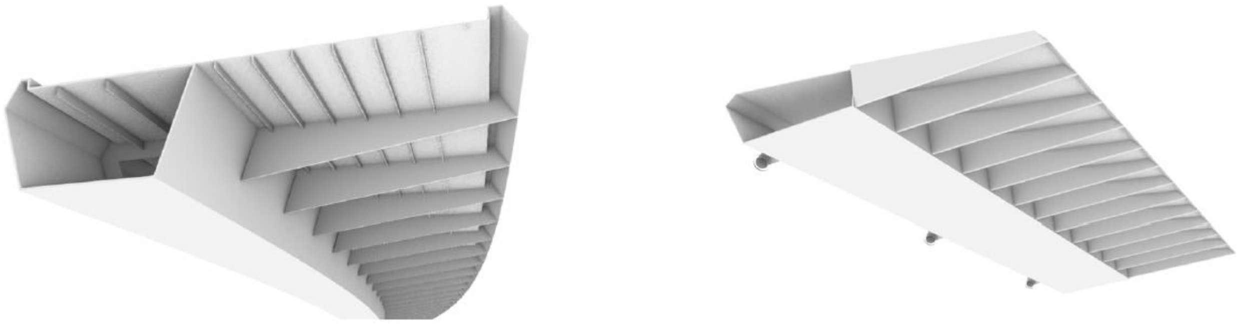

Figure 7: Deck section - from structural functional (left) to aesthetic pleasing (right)

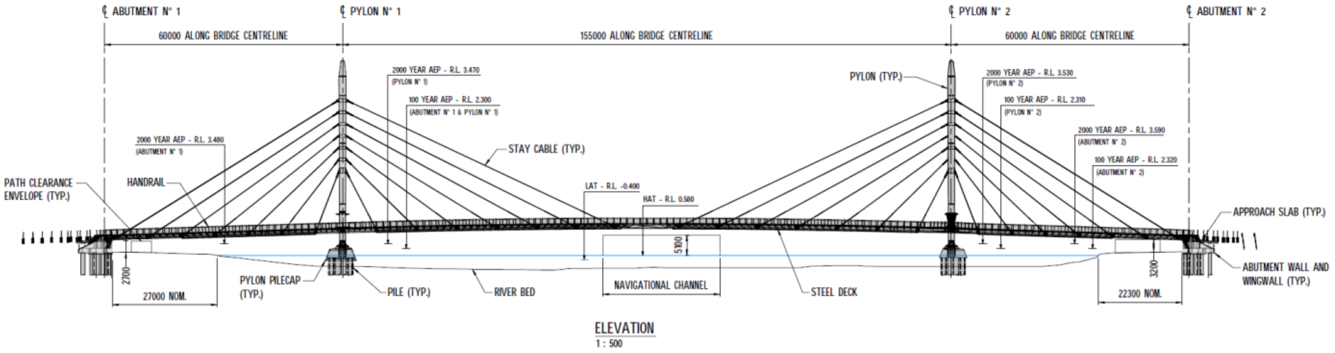

Figure 8: Developed elevation of McCallum Park Bridge

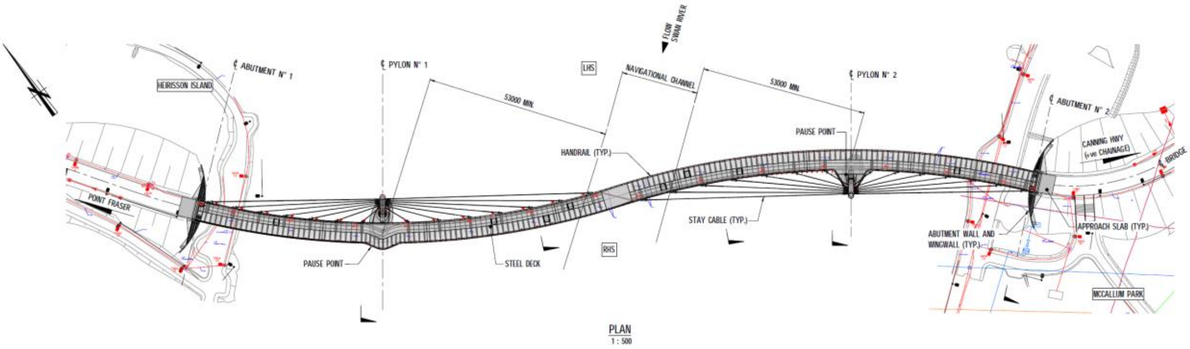

Figure 9: Plan view of McCallum Park Bridge, the pause points are visible at the pylons

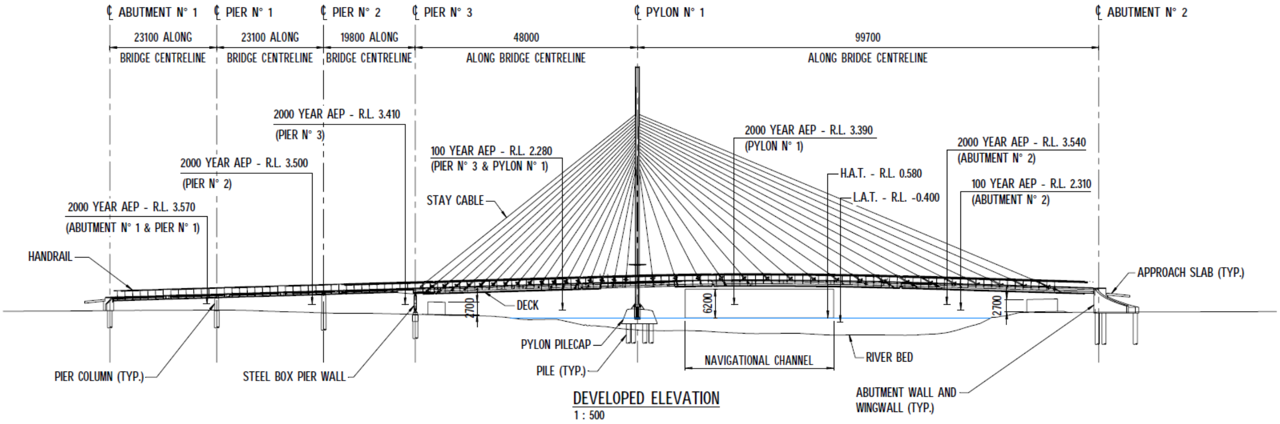

Figure 10: Developed elevation of Point Fraser Bridge

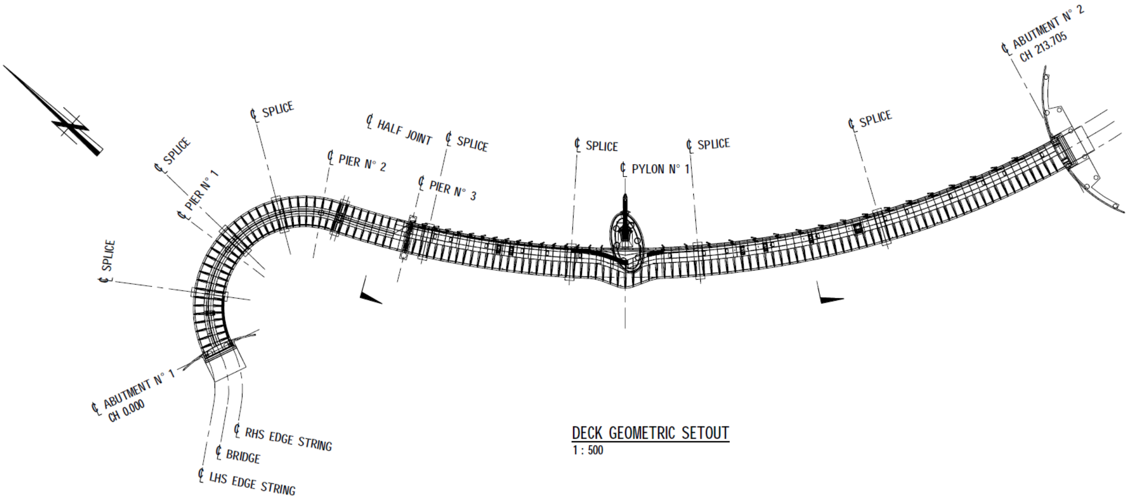

Figure 11: Plan view of Point Fraser Bridge, the point is at the pylon

The barriers and functional lighting

Functional lighting was developed as an integrated solution to meet user safety requirements while maintaining a visually lightweight barrier system. To avoid the use of light poles, luminaires were integrated within the handrail.

The barrier system was required to provide crowd restraint for pedestrians and provide safety for cyclists while maintaining transparency. The top rail is set at 1.4 m above the finished surface to comply with AS 5100 Part 1[1].

A smooth deflection rail for cyclists is provided at 1.2 m above the surface in accordance with Austroads Guide to Road Design Part 6A[2]. This rail also houses the handrail lighting to achieve the illuminance requirements of AS/NZS 1158.3.1[3]. The adopted barrier comprises Grade 400 weathering steel stanchions, Grade 316 stainless steel top and deflection rails, wire-rope mesh infill, and Grade 2205 stainless steel rails supporting the mesh (Figure 12). Stainless steel components are electrically isolated from the weathering steel to mitigate dissimilar-metal corrosion.

The feature-lighting

A stay-cable lighting system capable of displaying images and messages was evaluated during design development. Achieving the required LED density increased the number of stay cables on Point Fraser Bridge by a factor of three.

For McCallum Park Bridge, fabrication had commenced and the original cable arrangement was retained. A market review indicated that available integrated cable-lighting systems did not satisfy the project requirements; consequently, a bespoke fully integrated system was developed with the stay-cable supplier Freyssinet.

The system was configured with no protruding elements to avoid adverse aerodynamic effects. Design development included input from the project team and maintenance personnel and was supported by full-scale mock-ups.

The final installation comprises 17,130 LEDs and allows individual LED replacement.

The abutments

The abutments are cast in situ reinforced-concrete structures comprising a chamber that houses the deck support column and the deck pendulum anchor, together with wingwalls retaining the approach embankments. The chamber is enclosed by a backwall, sidewalls, and a curved front wall incorporating an access door.

Curved wingwalls extend from the front wall to retain the side slopes, and an approach slab is supported on the backwall.

Spherical bearings and pendulum anchors provide vertical support and torsional restraint; these components are concealed within the abutment chamber, with maintenance access provided via the front-face door.

Figure 12: Bridge barriers with integrated lighting and integrated stay cable lighting system

The pendulum anchors are designed for the full service life. Pins, bushes, and cheek plates are Grade 2205 stainless steel; the pendulum anchors and anchor plates are WR350L0-Mod400. Interfaces between dissimilar metals were coated to reduce electrical continuity and mitigate galvanic corrosion (Figure 13).

The approach spans

At the Point Fraser end, the abutment was further set back from the river edge than the minimum 20 m requirement by transitioning the main cable-stayed structure to three short approach spans. This reduced imported embankment fill quantities and increased available space for landscaping. It also reduced ground-improvement demands by limiting fill heights (generally <2.5 m), thereby reducing potential settlement impacts and interaction with the existing Causeway bridge abutment.

The asymmetric cable-stayed deck transitions to a symmetric configuration between Pier 2 and Pier 3 for the approach spans. To manage cyclist operating speeds and satisfy the maximum 3% longitudinal grade requirement, the approach spans incorporate a horizontal curve in the opposite direction to the main spans. This increases the overall bridge length but was adopted to provide a compliant and safe geometric solution.

Figure 13: Pendulum anchor to steel diaphragm connection

Because the deck curvature reverses through the transition, Pier 3 of the Point Fraser Bridge was detailed to accommodate longitudinal thermal movements, provide torsional restraint, and resist a portion of the back-span cable anchor forces. The adopted arrangement comprises a longitudinal pendulum anchor and a transverse diaphragm formed by a closed steel box (Figure 14).

The steel

Weathering steel was specified for the bridges as a project requirement, providing a low-maintenance external finish and reflecting regional material associations.

The initial design adopted WR350 weathering steel. To reduce self-weight while maintaining element sizes compatible with transport and lifting constraints, higher-strength weathering steel was required.

Figure 14: The approach spans and the deck transition at Pier 3

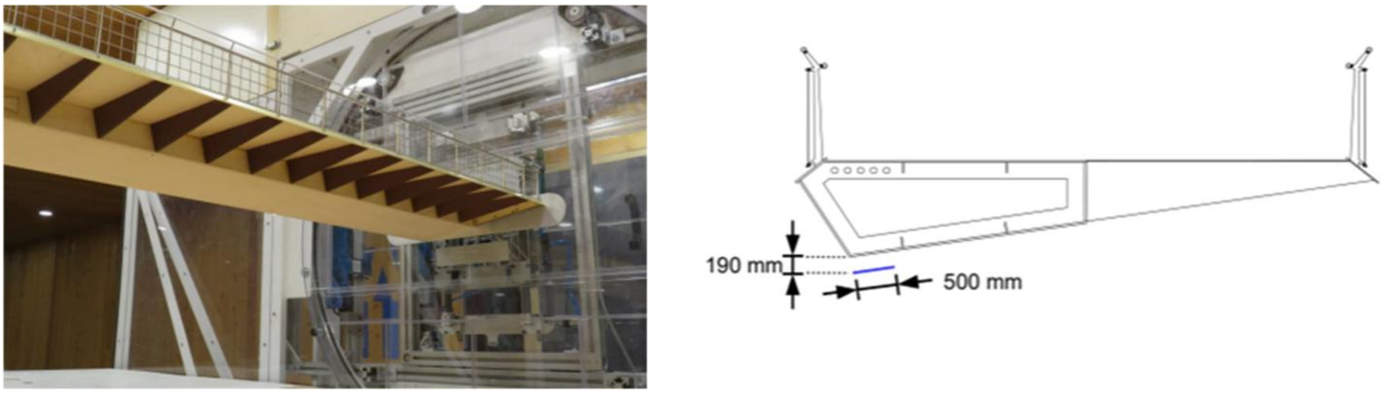

Figure 15: Sectional deck model in wind tunnel (left), wind splitter plate to mitigate VIO (right)

A 400 MPa-grade weathering steel developed by BlueScope (designated WR350L0-Mod400 for this project) was specified, noting that this grade is not yet included in the applicable Australian Standard. The L0 designation specifies impact-toughness testing at zero degree Celsius, which supports weldability and reduces the risk of micro-cracking in cold-formed outrigger components.

WIND DYNAMICS

Sectional models

Deck

Sectional-model wind-tunnel testing of the deck was undertaken by RWDI (Figure 15, left). Vertical vortex-induced oscillation (VIO) was observed for winds approaching from the box side, with a full-scale peak amplitude of approximately 80 mm. This exceeded the project comfort criterion (5%g acceleration for wind speeds up to 15 m/s), based on the first vertical modes of both bridges and an assumed structural damping ratio of 0.3% of critical.

No flutter or galloping was observed below the specified stability wind speed. Several mitigation options were evaluated; in consultation with the architects, a splitter plate was adopted (Figure 15, right), which eliminated the VIO response by suppressing unsteady vortex shedding.

Pylons

Separate wind-tunnel models of the pylons were used to measure mean forces and moments and to derive equivalent distributions of force coefficients. For the McCallum Park Bridge pylon, a high-Reynolds-number simulation was employed to represent the supercritical regime.

Base forces and moments reproduced using the derived coefficients agreed well with the measurements, providing confidence that the pylons would remain stable immediately after erection without requiring additional temporary bracing.

A full-bridge aeroelastic model was constructed at 1:55 scale. The model reproduced the mass and stiffness characteristics of the completed bridges; stay cables were represented to match full-scale mass and mean drag. Testing indicated that the deck and pylons remain stable against vortex shedding, galloping, and flutter at wind speeds exceeding the project stability criterion, defined as the wind associated with a 10,000-year return period.

Buffeting responses were measured in turbulent flow representative of the site for a range of wind directions; maximum responses occurred for skew wind directions relative to nominal bridge-normal winds, reflecting exposure differences and deck curvature. The revision to triple the number of stay cables on Point Fraser Bridge increased mean and dynamic responses due to increased cable drag; however, no global aerodynamic instability was observed.

Numerical wind buffeting analysis

A numerical wind-buffeting analysis was undertaken to validate the aeroelastic wind-tunnel results using aerodynamic force coefficients and flutter derivatives obtained from the sectional deck tests. The analysis was also used to determine displacements, accelerations, and force effects for critical scenarios not directly assessed in the aeroelastic testing.

Because the deck was supported by temporary piers during construction, it was not subjected to significant dynamic wind response in intermediate stages. The buffeting analysis therefore focused on the completed configuration. Separate desktop checks were undertaken to assess vortex-induced oscillation susceptibility of free-standing pylons during erection.

Wind parameters provided by the wind consultant (RWDI)—including the wind profile, turbulence characteristics, force coefficients, and flutter derivatives—were used as inputs. Drag, lift, and torsional moment per unit length were derived from the force coefficients, while motion-induced aerodynamic forces were represented using flutter derivatives.

A transient time-history approach was adopted. Artificial wind histories were generated incorporating aerodynamic damping and galloping effects. A representative ULS wind history at the top of a pylon, including along- and cross-wind components, is presented in Figure 16.

The analysis employed unsteady aerodynamic theory using flutter derivatives. For aerodynamic interaction, 10-minute simulations comprising 12,000 steps (0.05 s per step) were performed.

To improve statistical robustness, 11 simulations were run for each condition and the mean of the maximum responses was adopted.

An automated routine was developed to execute successive analyses for the investigated conditions. Response envelopes for displacements, accelerations, and force effects were generated for ULS and SLS design.

Close agreement between the aeroelastic wind-tunnel measurements and the numerical buffeting results was obtained, supporting the adopted modelling approach.

Dynamic amplification factors were derived by comparing static wind analysis results with the numerical buffeting analysis. These factors were applied to the static wind loading in the global structural analysis model for the final design.

Deck accelerations under operational wind speeds were also assessed against pedestrian comfort criteria. The calculated mean peak accelerations were within the specified limits, and no supplemental wind-damping devices were required for buffeting response control.

Cables

Stay-cable vibrations were assessed for direct and indirect excitation mechanisms, including wind-induced effects (buffeting and VIO) and deck vibration transmitted to the anchors. Frequency content of anchor vibrations was evaluated and components within ±20% of each cable natural frequency were used to assess the potential for resonance.

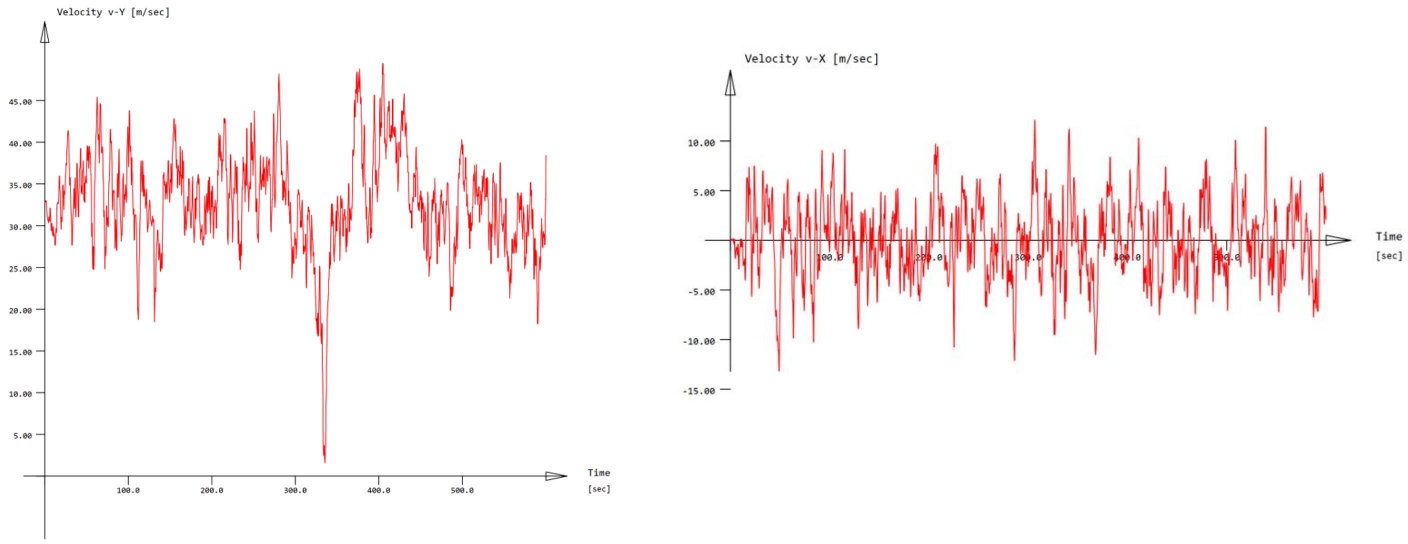

Figure 16: Left image: along wind history, right image: cross wind history

Cable response and parametric excitation were then checked in accordance with the Setra recommendations[6].

Project-specific vibration acceptance criteria were defined to limit perceptible motion for users and to mitigate fatigue demand in the cables and anchorage components. For mean wind velocities up to 15 m/s at deck level, allowable cable vibration amplitudes were set to L/500 and 1.0D, where L is the cable length and D is the cable diameter.

For the original Point Fraser Bridge cable arrangement, the analytical assessment indicated that external cable dampers were not required. The smaller number of cables resulted in relatively favourable combined mass damping characteristics, typically expressed using the Scruton number (Sc).

Following the increase in cable quantity and the introduction of integrated lighting, the stay-cable vibration assessment was repeated. The longest cables were identified as requiring internal dampers due to potential resonance between cable and global bridge modes.

The lighting system also influenced cable aerodynamics by increasing the effective diameter (reducing Sc) and introducing surface irregularities at the cable duct, which can increase susceptibility to certain instability mechanisms.

Figure 17: Testing of the integrated stay cable lighting system

The integrated lighting system was therefore developed to minimise adverse aerodynamic impacts.

To confirm aerodynamic performance, wind-tunnel tests were carried out by CSTB (France) on a 1:1 geometric-scale representation of the cable cross-section, including static and aeroelastic testing of a vertically inclined cable in an atmospheric boundary-layer wind tunnel (Figure 17).

Tests were performed under low structural damping and low turbulence; no rain-wind-induced vibrations were observed and VIO amplitudes (including regular VIO and high-reduced-velocity vortex shedding) remained small.

Dry inclined cable galloping was observed only at extreme wind speeds and under specific frequency conditions, namely when the vertical and lateral natural frequencies were nearly coincident (difference <1.6%).

For longer cables, sag effects increase the separation between these frequencies, reducing the likelihood of coincidence. The wind-tunnel results were extrapolated to the full cable set across both bridges, and internal dampers were specified for the seven longest Point Fraser Bridge cables to satisfy the stability criteria.

FOOTFALL DYNAMICS

Footfall design

For footbridges, vibration performance is typically governed by serviceability and user comfort.

A dynamic assessment was undertaken to quantify bridge response to rhythmic pedestrian and jogger loading and to determine whether the structures satisfy the specified comfort and stability criteria or require supplemental mitigation. The analyses followed the Eurocode framework and JRC Technical Report EUR 23984 EN[4].

The bridges were assessed for pedestrian crowd loading and for jogger loading representative of marathon events. Responses were evaluated against project-specific comfort classes defined by vertical and horizontal acceleration limits.

For pedestrian loading, the model in JRC Technical Report EUR 23984 EN[4] was adopted. While the report provides a load model for a single runner, it does not specify load models for large jogger groups.

Some National Annexes (for example, NA to BS EN 1991-2[5]) provide models for small groups (e.g., four joggers). For this project, jogger streams were statistically analysed to estimate an equivalent number of perfectly synchronised joggers for densities up to 0.5 joggers/m^2.

Lateral stability against pedestrian-induced lock-in was also assessed. In lock-in, pedestrians adjust their gait in response to lateral motion, increasing effective excitation near twice the bridge lateral natural frequency and potentially causing rapid amplitude growth in lightly damped modes.

JRC Technical Report EUR 23984 EN[4] presents (i) an approach by Dallard et al.[7] (based on the Millennium Bridge) to determine a triggering pedestrian number from damping ratio, modal mass, natural frequency, and a bridge constant, and (ii) an acceleration-threshold approach (typically at 0.1-0.15 m/s^2). Given the consequences of lock-in and the variability in pedestrian response, the more conservative Dallard-based method was adopted.

To satisfy comfort limits for walking and jogging and to provide stability margins against lateral lock-in, additional damping was required. Tuned mass dampers (TMDs) were adopted.

TMD Type	fd. (Hz)	fm (Hz)	ft (Hz)	fd. (Hz)	fm (Hz)	ft (Hz)
1	1.75	1.88	1.82	2.03	1.86	1.80
2	1.97	1.88	1.84	1.74	1.90	1.82
3	2.41	2.34	2.15	0.68	0.79	0.72
4	1.5	1.47	1.45	0.68	0.79	0.72
5	3.0	2.88	2.85	-	-	-
6	3.17	3.4	3.35	-	-	-

fd = Design natural frequency
fm = Measured natural frequency
ft = TMD tuning frequency

Table 1: Vertical and horizontal natural frequencies

Figure 18: Runners crossing the bridges from the McCallum Park side

Because TMD effectiveness is frequency-dependent and the critical excitation frequency ranges differ for walkers and joggers, separate dampers were provided for the respective target modes.

In total, 11 TMDs were installed on McCallum Park Bridge and 6 TMDs on Point Fraser Bridge; where practicable, vertical and horizontal dampers were combined to reduce overall mass.

Dynamic testing and commissioning of the TMDs

Both bridges were tested broadly in line with the procedures suggested in JRC Technical Report EUR 23984 EN[4] using a group of 15 participants. Measured natural frequencies were in good agreement with design predictions, enabling tuning of the TMDs within the specified tolerance ranges (Table 1).

Following installation and engagement of the TMDs, the tests were repeated to quantify damping performance. Measured accelerations were extrapolated to estimate the number of perfectly synchronised pedestrians required to exceed comfort criteria and to trigger lateral lock-in instability.

As part of the opening event, approximately 530 runners crossed the bridges as the first organised event (Figure 18).

Observations from this event were consistent with the predicted dynamic performance under combined pedestrian-induced and wind-induced vibration demands.

CONSTRUCTION SEQUENCING

Integration with adaptive, performance-based foundation risk management

Construction sequencing for the Boorloo Bridge was coordinated closely with the geotechnical delivery strategy, which adopted an adaptive, performance-based risk management framework to address the challenges of soft ground conditions, deep paleochannels, and strict deformation limits.

This framework, developed and implemented in parallel with the bridge design, combined principles from ISO 31000, the Observational Method, and performance-based design to enable responsive adaptation of foundation and ground-improvement solutions during both design and construction stages.

For the Boorloo Bridge, the framework facilitated early identification of sequencing-related risks—particularly those associated with surcharge loading, ground improvement, and piling near sensitive structures and culturally restricted areas.

Performance-based trigger criteria, supported by real-time monitoring and staged numerical modelling, allowed the construction sequence to be adjusted dynamically in response to observed ground behaviour and unforeseen subsurface conditions.

This approach reduced the need for overly conservative sequencing constraints, mitigated programme risk, and ensured that construction-stage behaviour remained within defined serviceability and stability limits while accommodating deviations from assumed ground models.

The adaptive framework proved particularly valuable during construction, where deviations from the planned sequence and unexpected ground anomalies required rapid reassessment and targeted interventions without compromising safety, long-term performance, or programme certainty.

Performance-based sequencing enabled by cracking-tolerant ground-improvement elements

A key enabler of the adopted construction sequence was the use of a performance-based ground-improvement design philosophy in which drilled displacement columns (also referred to as controlled modulus columns) were explicitly permitted to develop controlled flexural cracking during construction stages.

Rather than treating cracking as a failure condition, the columns were designed as geotechnical inclusions whose primary function—axial stiffness and settlement control—could be maintained despite localised cracking under transient bending demands induced by surcharging and parallel construction activities.

This cracking-tolerant design approach enabled a parallel construction sequence in which ground improvement, surcharging, abutment works, and associated earthworks could proceed concurrently, significantly reducing programme duration compared with a conventional linear sequence.

Numerical modelling and field validation demonstrated that, provided defined residual stiffness and deformation limits were respected, controlled cracking did not compromise settlement performance, differential movement control, or overall embankment stability.

Selective local reinforcement was introduced where required to prevent column dislocation under peak construction-stage bending, while still preserving the intended crack-tolerant behaviour.

This performance-driven strategy aligned with the broader adaptive risk framework and provided schedule resilience during construction, particularly in areas where access, cultural constraints, or evolving ground conditions limited flexibility in staging.

Bridge construction

The adopted construction scheme assembled prefabricated deck modules (up to approximately 45 m long) on temporary pier supports. Segments were spliced at temporary support locations using bolted connections to enable rapid assembly, followed by permanent welded splices.

Stabbing guides formed from circular hollow sections were used to facilitate alignment of incoming modules (Figure 19).

The deck was assembled prior to installation of the pylon link beams, followed by stay installation and a two-stage stressing procedure.

In the first stage, stays were stressed to approximately 10-15% below final target forces. Temporary supports were then lowered and the resulting cable forces were evaluated, allowing adjustments before the second (final) stressing stage to achieve the specified force and geometry targets.

Detailed staged-construction analysis was undertaken to confirm that structural behaviour during erection aligned with design assumptions and to quantify force accumulation at each stage and the final force distribution.

Cable force finding was integrated with the staged analysis using an iterative solver to meet target force distributions in the deck and pylons. The analysis included geometric nonlinearity (second-order effects) and cable sag.

Moderate vertical and torsional pre-cambers were introduced in deck modules to achieve the final profile; pre-camber of the pylons was not required due to limited final deflections.

Locked-in sagging moments developed due to splicing at temporary support locations. To mitigate these effects, the deck end remote from the splice was initially set at a higher temporary elevation.

After bolting the splice, the segment was lowered, reducing sagging moments and introducing compensating hogging effects, thereby improving the overall force balance prior to welding.

Overall, the construction scheme and associated staged analysis enabled efficient module erection while achieving the required final geometry and structural performance.

Barge-mounted cranes were initially considered for module installation; however, tidal influences and associated water-depth limitations constrained the feasible barge and crane sizes.

The installation methodology was therefore revised to use land-based crawler cranes.

To avoid changes to segment sizes and sequencing, a high-capacity crawler crane was deployed, enabling the installation of approximately 110 t segments with a reach exceeding 100 m. This required the construction of substantial crane pads, located with sufficient clearance from the abutments to avoid adverse effects on the foundations.

CONCLUSION

This paper presented the design and verification of two curved cable-stayed active-transport bridges spanning the Swan River in Perth, providing a 6.0 m wide segregated shared path between East Perth and Victoria Park via Heirisson Island.

Key design-development outcomes included the adoption of an asymmetric, torsionally stiff steel box deck to accommodate single-sided stay anchorage on curved alignments, the refinement of pylon and deck details to satisfy functional and stakeholder requirements, and the selection of 400 MPa weathering steel to reduce self-weight while meeting fabrication and erection constraints.

Wind engineering assessment combined sectional and aeroelastic wind-tunnel testing with numerical buffeting analysis. A splitter plate was adopted to mitigate deck vertical vortex-induced oscillation identified in sectional testing.

Figure 19: Stabbing guides to efficiently align and join bridge segments with the receiving unit on the left and guides on the right module

Figure 20: 1600-ton crawler crane lifting segment 5 of McCallum Park Bridge

Pedestrian-induced vibration performance was assessed for crowd and jogger scenarios; tuned mass dampers were installed and commissioning tests indicated that the specified serviceability criteria were satisfied.

On Point Fraser Bridge, an increased number of stay cables supported an integrated lighting system. Development of the system required aerodynamic validation of the modified cable surface and provision for maintainability, including individual LED replacement.

Future work could include long-term monitoring of wind- and crowd-induced response to further calibrate damping assumptions and to inform optimisation of similar cable-integrated systems.

ACKNOWLEDGMENTS

The Causeway Link Alliance (CIVMEC, Seymour Whyte, and WSP) delivered the project under an Alliance Agreement with Main Roads Western Australia. The project was jointly funded by the Australian and Western Australian Governments as part of the Perth City Deals.

The Matagarup Elders Group contributed to concept development and shared cultural narratives that informed the design and delivery process.

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Figure 21: Point Fraser Bridge displaying Yagan

THE MOLONGLO RIVER BRIDGE CANBERRA, AUSTRALIA

PROJECT INTRODUCTION

The Molonglo River Bridge is a 200-metre-long superstructure under construction in Canberra, Australia. Once complete, it will be one of Australia's longest weathering steel bridges and the tallest road bridge in Canberra.

Australia's largest privately owned civil construction company, **BMD**, is delivering the Molonglo River Bridge project for the **Federal and Australian Capital Territory (ACT) governments**.

The project represents a significant piece of infrastructure, combining complex structural geometry, major lifting operations and innovative construction methodologies within a constrained and environmentally significant river corridor.

Central to this complexity is the bridge's composite structural system, which comprises braced pairs of variable-depth weathering steel trapezoidal box girders supporting precast concrete deck panels and an overlaid in-situ reinforced concrete deck.

To develop and deliver the design, **BMD engaged pitt&sherry, its sub-consultant COWI, and GHD** for the associated civil works, bringing together a multidisciplinary team to help deliver the project and its architectural outcomes.

The project forms part of the **John Gorton Drive extension**, completing a critical north–south transport corridor connecting Canberra's rapidly growing Molonglo Valley to the broader ACT road network. The project replaces the flood-prone low-level crossing at Coppins Crossing Road, providing a safer, more reliable connection that remains operational during major flood events while accommodating road traffic, active transport and future light rail infrastructure.

Located within the Molonglo River Reserve, an area of national environmental significance, the project traverses a sensitive river valley that supports platypus, waterfowl and other protected fauna.

The bridge design was therefore required to achieve multiple objectives: improving transport resilience and connectivity, minimising environmental impacts, integrating with the surrounding landscape and providing long-term adaptability for future growth.

Through close collaboration between the client, contractor, designers and proof engineers, the project evolved from the reference concept to a highly efficient and constructible solution that balanced structural performance, flood resilience, maintenance requirements, sustainability outcomes and architectural intent.

The following articles outline the development of the bridge design and construction methodology, highlighting key innovations that enabled the successful delivery of the project to date. They include drawings and the Construction Gallery.

DESIGNING FOR RESILIENCE, CONSTRUCTABILITY AND PLACE: THE MOLONGLO RIVER BRIDGE, CANBERRA

Irene Scott, Executive Director / Senior Principal Engineer / General Manager Bridges and Structural Engineering

pitt&sherry

Figure 1: Construction of the Molonglo River Bridge

PROJECT CONTEXT AND DESIGN DRIVERS

Strategic Role and Network Function

The Molonglo River Bridge is the final crossing required to complete John Gorton Drive as a 7.2 km arterial route linking Cotter Road in the south to William Hovell Drive in the north.

When complete, the corridor will support private vehicles, buses, emergency services, cyclists, pedestrians, and future light rail, serving a catchment exceeding 70,000 residents.

From a network perspective, uninterrupted serviceability under flood conditions was a primary driver.

The bridge alignment and vertical geometry were developed to provide substantial flood immunity, ensuring continuity of access for public transport and emergency response during extreme events.

Environmental and Architectural Context

The bridge traverses the Molonglo River Reserve, a landscape characterised by rugged rock formations, open grasslands, and riparian vegetation.

Recognising the high visual exposure of the crossing, the ACT Government established architectural parameters requiring the bridge to integrate with its surroundings and align with a broader "Family of Bridges" concept.

Weathering steel — already used on the adjacent Butters Bridge — was adopted as a unifying material, providing visual consistency with the landscape while minimising maintenance requirements.

Environmental considerations influenced decisions ranging from pier placement (outside the normal watercourse) to construction methodology, with a strong emphasis on reducing in-river works and disturbance to fauna habitats.

PROCUREMENT MODEL AND DESIGN DEVELOPMENT

Develop, Design and Construct Delivery

The project was delivered under a Design & Construct (D&C) model in which the ACT Government developed the concept design to the Preliminary Sketch Plan (PSP) stage and secured key environmental and statutory approvals prior to engaging a delivery partner. This approach reduced project risk and allowed the D&C team to focus on optimisation rather than re-establishing fundamentals.

This early contractor involvement enabled buildability and construction methodology to directly inform structural decisions. The equitable sharing of risk between client and contractor created an environment conducive to meaningful innovation, rather than conservative replication of the reference design.

Evolution from Reference Design

The PSP reference design adopted a 227 m long bridge with an unequal 60–93–72 m span arrangement and three narrow rectangular box girders per carriageway. While technically feasible, this arrangement was materially intensive, less efficient structurally, and challenging from both construction and maintenance perspectives.

Figure 2: Location of the Bridge
Click on the image to open Google Maps

During the tender and the early detailed design phase, the alignment was refined to cross the river more orthogonally, enabling a reduced bridge length of 200 m and a rationalised 60–80–60 m span arrangement. This change delivered measurable savings in material quantities, simplified construction staging, and provided improved maintenance access.

STRUCTURAL FORM AND SUPERSTRUCTURE DESIGN

Selection of Steel Box Girders

Steel box girders were selected as the preferred superstructure form due to their high torsional stiffness, clean architectural lines, and suitability for longer spans.

Compared with plate girder alternatives, box girders offer improved durability through reduced exposed surface area and fewer corrosion-prone details.

The final configuration comprises twin trapezoidal weathering steel box girders per carriageway, made composite with a reinforced concrete deck slab.

The trapezoidal form was selected to maximise structural efficiency, allowing the deck slab to contribute effectively to transverse load distribution while limiting cantilever lengths.

Geometry and Structural Efficiency

Each box girder varies in depth from approximately 2.0 m at the abutments to a 4.0 m haunched section over the piers. This variable depth significantly reduces mid-span steel tonnage while maintaining stiffness and deflection performance.

The transition from three narrow boxes to two wider boxes delivered multiple benefits:

- Reduced steel quantity and embodied carbon
- Fewer girder segments and lifting operations
- Fewer splice connections
- Improved global stability during erection
- Increased internal clearance facilitating inspection and maintenance

These refinements collectively enhanced both construction safety and life-cycle performance.

DURABILITY AND MATERIAL STRATEGY

Weathering Steel and Fracture Control

All primary steelwork is fabricated from WR350 weathering steel. While weathering steel offers long-term durability advantages through the formation of a protective patina, its application must be carefully matched to environmental exposure and fracture risk.

Australian Standard AS 5100.6 adopts conservative brittle fracture provisions based on legacy British standards, which would have mandated higher sub-grades for a substantial proportion of the girder plates.

A project-specific brittle fracture study was undertaken, benchmarking AS 5100 requirements against current Eurocode practice.

The study demonstrated that Eurocode EN 1993-1-10 provisions were technically appropriate for the Molonglo River Bridge's design temperature and inland exposure.

With ACT Government endorsement, steel up to 65 mm thickness was specified as WR350-L0, with L20 reserved only for thicker plates. This approach reduced material cost and procurement complexity without compromising safety.

Deck and Drainage Detailing

The composite deck comprises precast concrete panels acting as permanent formwork, topped with an in-situ slab. This approach improved construction efficiency while maintaining high quality control.

To enhance durability and reduce future maintenance risk, through-deck drainage systems and internal scuppers were eliminated.

All deck drainage is managed longitudinally along the deck surface, avoiding penetrations through the box girders and eliminating a common source of corrosion and leakage in steel bridges, as well as the risk of flooding due to blocked drains.

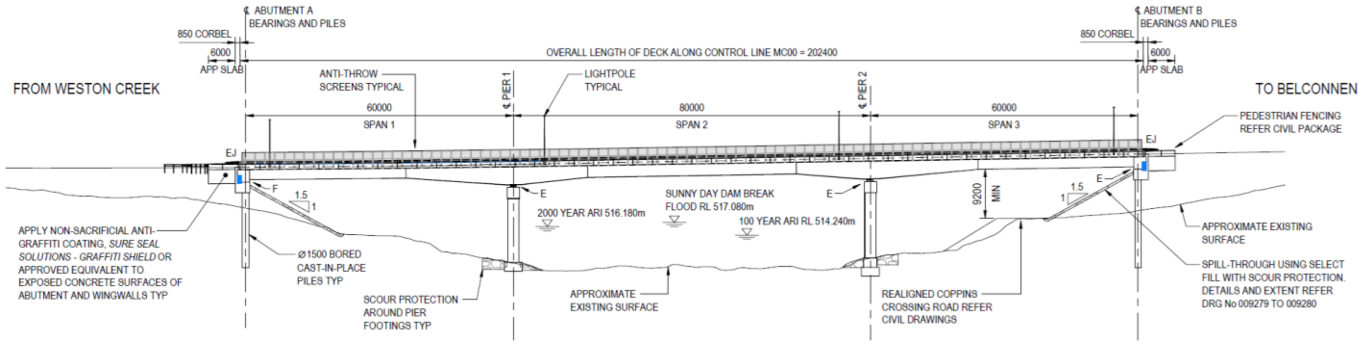

Figure 3: Elevation of the Bridge
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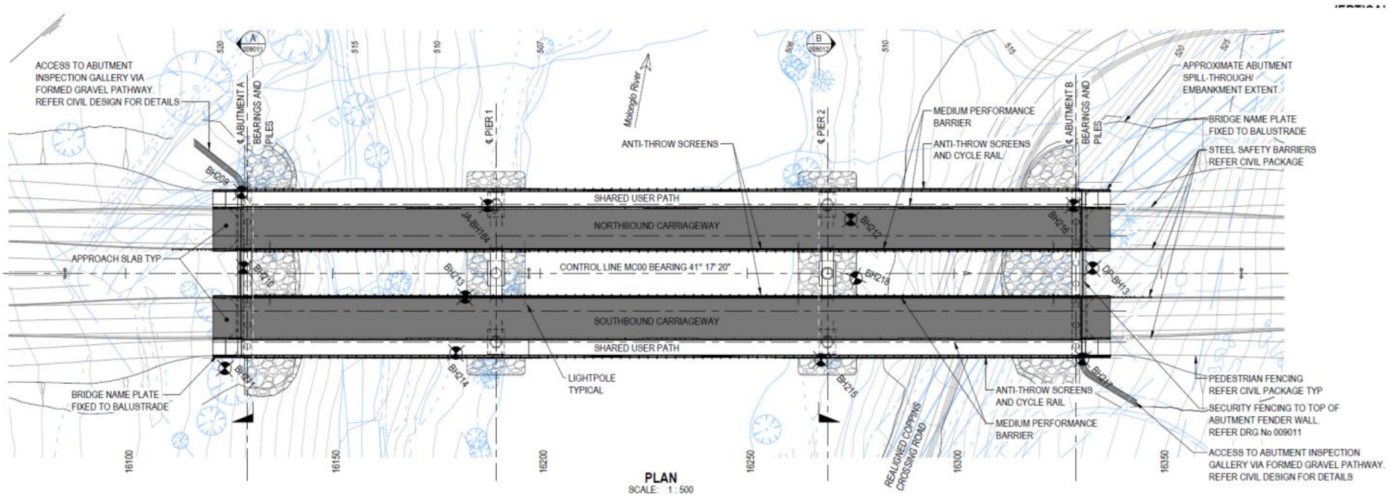

Figure 4: Plan
Click on the image to open it in a higher resolution

FLOOD, DAM-BREAK AND EXTREME EVENT DESIGN

Flood Design Philosophy

Flood resilience was a governing design consideration. The Molonglo River is influenced by Scrivener Dam upstream, requiring assessment not only of conventional flood events but also a sunny-day dam-break scenario.

The bridge was designed to withstand:

- 1:2000-year Average Recurrence Interval (ARI) flood events
- Sunny-day dam-break flood loading without loss of structural integrity

Piers are founded in bedrock and positioned outside the normal flow channels. Extensive hydraulic modelling informed pier shape, orientation, and scour protection, reducing drag forces and minimising debris accumulation.

Scour and Freeboard

Scour protection comprises engineered rock protection around both piers and abutments, with depths determined by detailed flood modelling and geotechnical assessment.

The vertical clearance between the soffit and extreme flood levels was provided to ensure additional resilience, reducing uplift and impact risk during these rare events.

PROVISION FOR FUTURE LIGHT RAIL

A defining feature of the Molonglo River Bridge is the provision for a future 10 m wide light-rail bridge deck located between the road carriageways.

Rather than retrofitting capacity, the substructure was designed from the outset to accommodate the additional permanent and live loads associated with light rail.

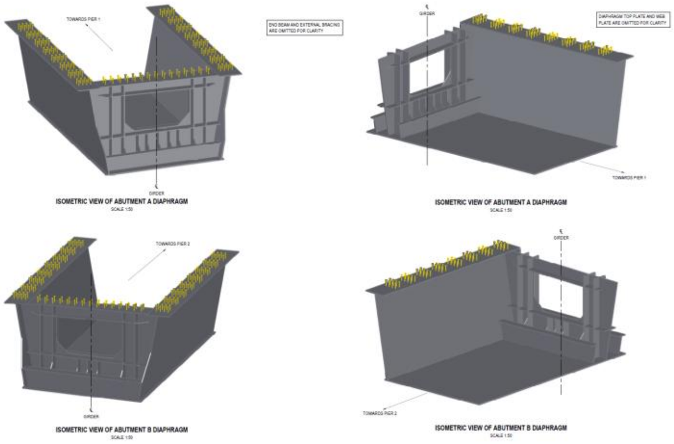

Figure 5: Isometric view of the steel box girders at the abutment
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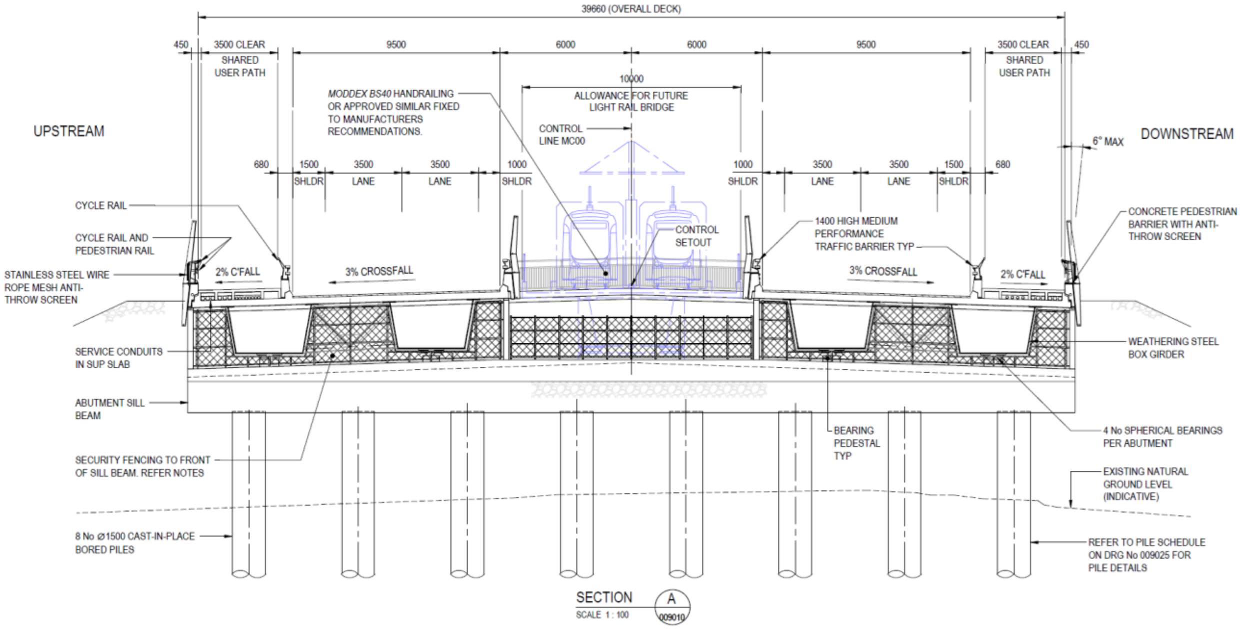
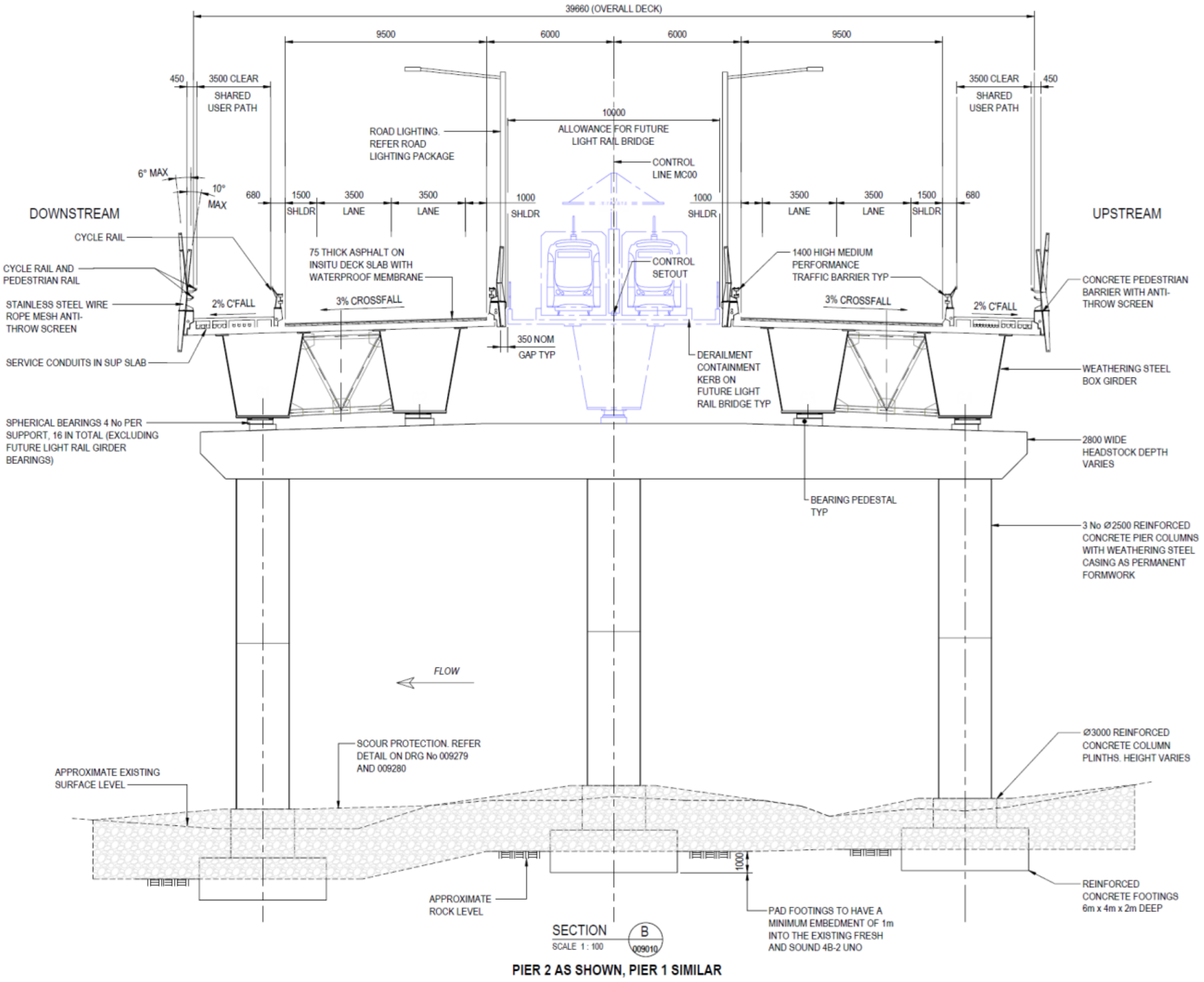




Figures 6 and 7: Sections at abutment and pier, showing the future light-rail location
Click on the image to open it in a higher resolution

Key aspects include:

- Pier headstocks sized and arranged to directly support future rail girders
- Abutment foundations designed for both road and rail load combinations
- Road bridge decks verified for construction loads associated with future light rail girder erection

Importantly, the future rail superstructure can be installed from the existing bridge decks, avoiding disturbance to the river corridor and reserve. This foresight significantly reduces future environmental and construction impacts.

CONSTRUCTABILITY AND GIRDER INSTALLATION

Construction Method Selection

The ACT Government stipulated that long-term closures of Coppins Crossing Road were not permissible during bridge construction. Following extensive optioneering, a heavy-lift crane methodology was selected over incremental launching.

This decision was influenced by:

- Reduced temporary works complexity
- Lower risk to permanent works
- Reduced in-river activity
- Improved construction certainty

Heavy-Lift Installation

Steel girders were fabricated in 40 m segments, transported to site, and pre-spliced into 80 m assemblies. Installation was undertaken using a 1,600 t Terex Demag CC8800-1 crawler crane — one of only two such heavy-lift cranes operating in Australia.

The installation sequence required the crane to be mobilised on both sides of the river, with three distinct lift configurations.

Precision placement was achieved using a synchronised lifting system, enabling millimetre-level adjustments in grade and crossfall during installation. This capability was critical for the successful mid-span closures and bolted splicing at height.

INSPECTION, MAINTENANCE AND WHOLE-OF-LIFE PERFORMANCE

A strong emphasis was placed on long-term operability. Abutment inspection galleries provide a direct, safe access to the interiors of the box girders via a door in the end diaphragm. The internal spaces are ventilated and detailed to avoid classification as confined spaces, significantly improving inspection safety.

All elements with design lives less than 100 years — such as bearings and expansion joints — have been detailed for replacement without disruptive structural intervention.

Figures 8 and 9: Installation of the midspan girder and the deck panels

LESSONS LEARNED

The delivery of the Molonglo River Bridge highlights several lessons that are applicable to major bridge projects operating in sensitive environments, constrained corridors, and evolving transport networks.

While the project benefited from a clear strategic vision, its ultimate success was driven by the willingness of the client, contractor, and design team to challenge reference concepts and adopt evidence-based refinements during detailed design.

Early Investment in Alignment and Span Rationalisation Pays Dividends

One of the most critical decisions on the project was the refinement of the bridge alignment to cross the Molonglo River more orthogonally. This enabled a reduction in overall bridge length and the adoption of a rationalised 60–80–60 m span arrangement.

Although modest, this change significantly improved structural efficiency, reduced material quantities and simplified both construction staging and long-term maintenance.

The lesson is that early geometric refinement — particularly in river crossings — can unlock disproportionate benefits in design, construction, and sustainability outcomes.

Structural Efficiency and Constructability Are Interdependent

The transition from three narrow rectangular box girders to two wider trapezoidal box girders per carriageway was driven not only by structural efficiency but also by constructability and safety considerations. Wider box girders provided improved torsional stiffness and better utilisation of the composite deck slab, while simultaneously reducing the number of girder units, lifts, and splice locations.

This decision also enabled safer construction processes and significantly improved access for inspection and maintenance.

Heavy-Lift Construction Can Be Preferable to Complex Launching in Sensitive Environments

Although incremental launching was initially considered, the final adoption of a heavy-lift crane methodology proved advantageous given the project constraints.

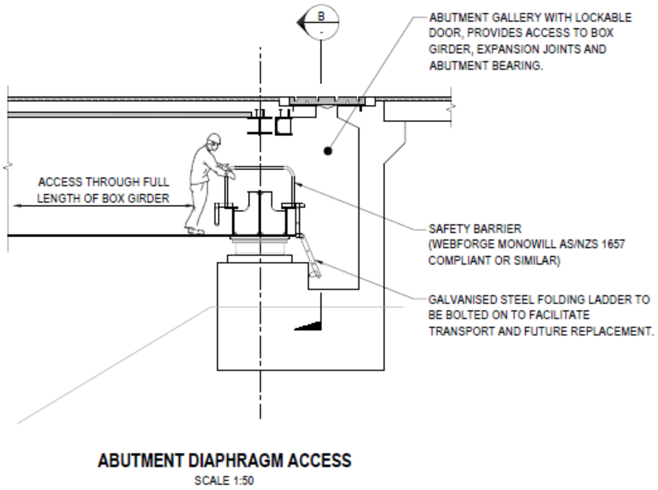


Figure 10: Abutment diaphragm maintenance access

The ability to install 80 m girder segments using a large crawler crane reduced temporary works complexity, avoided permanent design compromises associated with launching loads, and limited disturbance within the Molonglo River Reserve.

A key lesson is that while incremental launching can be attractive for long spans, heavy-lift solutions should not be discounted — particularly where environmental sensitivity, programme certainty and structural simplicity are dominant drivers.

Future-Proofing Is Most Effective When Integrated, Not Retrofitted

Provision for a future light-rail crossing was embedded into the bridge substructure and articulation strategy from the outset.

Designing pier headstocks, abutments, and bearings to accommodate future rail loads avoids the need for future complex retrofit solutions.

Importantly, the ability to construct the future light-rail superstructure from the road bridge decks, without entering the river corridor, represents a significant long-term environmental and constructability benefit.

The project reinforces that genuine future-proofing must be planned holistically at system level, rather than applied as an allowance late in design.

Designing for Inspection and Maintenance Improves Whole-of-Life Safety

The incorporation of abutment inspection galleries, non-confined internal girder spaces and accessible bearing replacement systems reflects a conscious shift toward designing with maintenance crews in mind.

Eliminating common operational hazards and simplifying access arrangements delivers long-term safety benefits that far outweigh the modest increase in upfront design effort.

This project reinforces the importance of treating inspection and maintenance as core design criteria rather than post-design considerations.

Collaboration and Governance Enable Innovation

Finally, the project demonstrates the value of strong governance frameworks — including design traceability, Safety-in-Design processes, independent proof engineering, and structured departures management — in enabling innovation rather than constraining it.

The Develop, Design and Construct model, combined with early contractor involvement and a shared focus on "best-for-project" outcomes, allowed the design team to simplify the structure, reduce risk, and improve sustainability without compromising safety or compliance.

CONCLUSION

The Molonglo River Bridge illustrates how integrated design thinking — spanning alignment, structural form, constructability, resilience, future transport provision, and whole-of-life performance — can deliver infrastructure that is both technically robust and contextually sensitive.

The lessons from this project are directly transferable to major bridge projects operating in flood-prone, environmentally sensitive, or rapidly developing urban corridors.

Figure 11: Heavy lifting operation at sunset

Molonglo River Bridge, Canberra, Australia

CONSTRUCTING THE MOLONGLO RIVER BRIDGE, AUSTRALIA

Joel Zuccato, Project Engineer

BMD CONSTRUCTIONS

Figure 1: Lifting of a girder

THE BRIDGE

The Molonglo River Bridge Project in Canberra represents a significant piece of infrastructure, combining complex geometry, heavy-lift operations, and innovative construction methodologies within a constrained and environmentally significant river corridor.

The 200m long bridge features a composite structural design comprising of braced pairs of variable-depth, weathering steel trapezoidal box girders. These girders support a series of precast concrete panels and an overlaid in-situ reinforced concrete deck.

Delivery of the bridge required careful integration of the permanent works, sophisticated temporary works, high-capacity lifting operations and complex logistics challenges, all of which were planned with extensive detail.

SUBSTRUCTURE WORKS

Construction commenced with the reinforced concrete substructure, which consists of abutments and two intermediate piers.

The abutments are each supported on eight 1500mm diameter bored piles installed to a depth of up to 20 m and socketed into the underlaying high-strength dacite bedrock.

The abutment design allows for future Government maintenance access, including a gallery walkway for internal girder entry and bearing inspections.

The piers are comprised of 2.0m thick reinforced concrete pad footings, reinforced concrete columns encased in a permanent weathering steel pier column liner, and a reinforced concrete headstock common to separate carriageway superstructures.

The reinforced concrete pier foundations are embedded into the extremely high strength dacite found in the river bedrock. Excavation of the footings involved extensive mechanical works using 50t excavators with hammers, rock drilling rigs, and a series of back blinding layers used to prevent water ingress and protect the structural elements during and after excavation from the adjacent river.

Each pier has three reinforced concrete pier columns that support a common 40m wide headstock. Each column is 2500mm in diameter and wrapped in an architectural weathering steel casing. The overlaying headstock is approximately 750 tonnes and is designed to support future light rail opportunities.

Construction of the pier columns and headstock involved designing a sophisticated temporary support structure, which was used to support the in-stage construction loads, as well as providing rigidity to the structure during intermittent flood events. This temporary support structure was founded on the pad footings and after pier column construction, braced against the same, thereby ensuring stability of the temporary work even during river flood events.

HEAVY-LIFT GIRDER INSTALLATION

The installation of the weathering steel box girders was one of the most technically demanding phases of the project. Girder segments were fabricated offsite and delivered to site on a three-day journey under full police escort. The superstructure comprises four long-span trapezoidal box girders split into five individual segments, connected via bolted splice.

One of Australia's largest operational crawler cranes, the DEMAG CC8800-1, with a rated capacity of 1,600 tonnes, was mobilised to the site from Western Australia for these operations, utilising 125 semi-trailer loads over a three-week period. The main girder lifts included eight spliced segment lifts weighing approximately 315 tonnes; installed to a radius of 62 m, along with four midspan lifts of approximately 125 tonnes placed to a radius of up to 115 m. An in-situ double splice was completed at midspan to finalise girder continuity.

Extensive planning was undertaken to manage transport logistics, onsite storage, splicing, crane movements and lift sequencing. Precision during installation was critical to ensure correct girder geometry, bearing alignment and splice tolerances.

All girder lifts were completed successfully without incident, achieving millimetre-level placement accuracy and maintaining program certainty. Planning for the girder lifts commenced in detail over 1 year prior to the operation, and the success of the girder installation phase vindicated the effort and attention paid to safety in completing this task.

Figure 2: Pouring concrete

Figure 3: Heavy lifting operation

A key logistics strategy on the project was to complete the splicing of the haunch and abutment segments on land. This allowed for tighter control of geometry, safety, and quality. This operation was underpinned by a coordinated temporary works system, including A-frame supports for haunch segments and adjustable concrete tower systems with jack and sledge plate assemblies, enabling precise alignment, replication of the longitudinal grade, and controlled management of thermal movement during assembly.

Once spliced, the combined element weighed up to 270 t and was transferred using self-propelled modular transporters (SPMTs), whose multi-axle, independently controllable configuration allowed the segments to be manoeuvred with high precision from storage and splicing areas into the crane pick zone.

Custom support frames and transport cradles ensured stability during movement and compatibility with both storage and lifting arrangements, creating a seamless link between assembly and installation.

Midspan installation represented the most challenging stage of the erection, requiring a tailored solution to minimise time on the hook while enabling safe splicing.

Shear beams were installed to temporarily suspend the midspan from the erected pier segments, allowing partial bolting to be completed before the crane was released.

To facilitate installation, the Abutment B spliced segments were initially jacked back by approximately 100 mm to create additional clearance for positioning the midspan, before being jacked forward to close the gap and achieve splice alignment. This combination of temporary suspension and controlled jacking enabled efficient installation, with full splice completion carried out progressively after the lift.

BEARING INSTALLATION AND LOAD TRANSFER

Sixteen spherical bearings were installed on the completed headstocks to support the steel girders. Bearing installation involved precise survey set-out, controlled placement and grout application to ensure full bearing contact under load transfer. Following girder installation, load transfer onto the bearings was carefully monitored and executed without incident.

Another key challenge during installation was managing the thermal behaviour of the girders during bearing alignment and ensuring proper load transfer under the changing ambient conditions.

As the girders expanded and contracted throughout the day, relative movement often caused misalignment between the girder and the bearing bolt holes. Temporary works were specifically designed to accommodate this behaviour, incorporating greased Teflon sliding pads that allowed the girders to expand and contract freely in the longitudinal direction without restraint.

Figure 4: Installation of the last girder

Figure 5: View from below

Rather than forcing connections, the team adopted a controlled approach, allowing the structure to naturally return to alignment within tolerance before completing the bearing fixings.

PRECAST DECK PANEL INSTALLATION

The concrete deck system was formed using precast reinforced concrete panels installed between the steel girders.

A total of 414 panels were installed, forming permanent formwork for the in-situ deck pours. Panel installation was carried out using a combination of a 275-tonne and an 800-tonne crawler crane, with some panels installed at a radius of up to 160 m.

Panel placement required close coordination among crane operations, survey control, and deck sequencing to maintain line, level, and crossfall tolerances. Temporary works and lifting methodologies were developed to suit the varying panel sizes and locations across the bridge span.

Due to unfavourable wind conditions between August and October, all 800-tonne crane operations were carried out at night, when wind speeds were more favourable.

SUPERSTRUCTURE DECK CONSTRUCTION

Following panel installation, reinforcement was placed and tied to form the in-situ concrete deck. Deck construction was staged into multiple pours to ensure symmetrical loading of the structure.

Survey analysis was undertaken prior to each pour to confirm deck thickness and reinforcement cover requirements, accounting for girder deflections and localised geometry variations.

Concrete placement, finishing and curing were tightly controlled to ensure durability, structural performance and finish quality.

Innovative access solutions, including a purpose-designed screed access gantry mounted on the screed rails, were developed to allow safe and efficient finishing works where conventional walkways were not feasible due to geometric and design constraints.

CONCLUSION

The Molonglo River Bridge Project demonstrates the successful integration of complex substructure works, high-capacity heavy lifting, precast deck construction and innovative temporary works solutions.

Through detailed planning, precise execution, and close collaboration between the engineering and construction teams, the project has delivered a technically robust structure while maintaining safety, quality, and program certainty in a challenging river corridor environment.

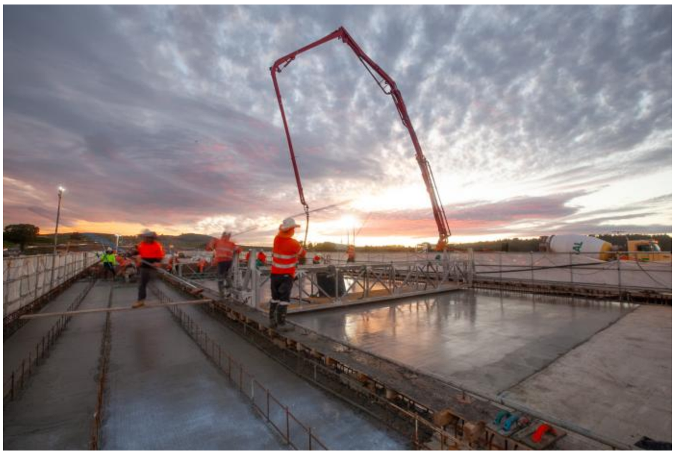
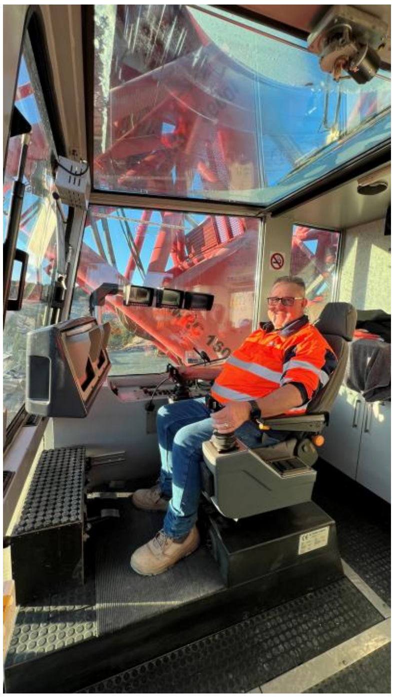
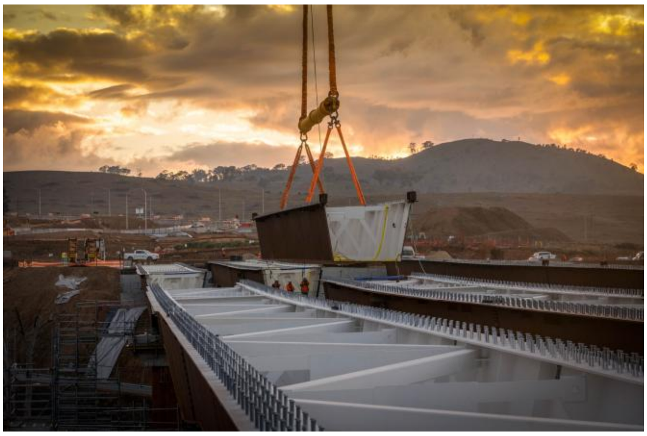


Figure 6: Pouring the deck

Figure 7: View of complete bridge

CONSTRUCTION GALLERY



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A FAUNA BRIDGE ON THE WILMAN WADANDI HIGHWAY – ACHIEVING SUSTAINABILITY AND HABITAT CONNECTIVITY

Dr. Dominique Cavell, Structures Lead
Nicholas Keage, Package Lead
South West Gateway Alliance

Figure 1: Yalinda Drive Bridge and connections to Fauna rope Structures

PROJECT SUMMARY

The Wilman Wadandi Highway is a 27-kilometre highway linking Forrest and Bussell Highway to create an alternative route around Bunbury, Western Australia's second largest city in the South-West of Western Australia.

The highway was opened in December 2024, easing congestion and providing safer, more efficient traffic routes for freight, tourists, and locals.

The project created unprecedented economic and social benefits for the region.

INTRODUCTION AND SITE CONTEXT

The Yalinda Drive Bridge carries Yalinda Drive (a local road, Figure 2), a shared path for pedestrians and a five-metre-wide fauna corridor, over the Wilman Wadandi Highway.

It is in close proximity to Western Ringtail Possum habitat and was designed with a strong focus on sustainability and minimising tree clearing as far as possible.

Fauna Rope structures are required to provide a safe means for Western Ringtail Possums to move safely across the new highway.

This mixed-use and adaptable bridge required screen walls, retaining walls, footpaths, and fauna rope structures to facilitate the movement of the various 'users' of this unique bridge over the road corridor.

BRIDGE DESIGN

The bridge is a simply-supported single-span pre-stressed concrete bridge, comprising 4 No. 2200 millimetre deep pre-tensioned precast concrete Teeroff beams with a composite in-situ reinforced concrete deck slab.

The mechanically stabilised earth (MSE) abutments and wingwalls comprise full height precast concrete facing panels. The bridge superstructure is supported independently of the abutment MSE walls on rectangular reinforced concrete columns, founded on reinforced concrete strip footing.

The bridge design includes consideration of SM1600 traffic loads occupying five standard design lanes across the full width of the bridge. The bridge was also designed to support a 10-tonne truck to account for emergency or maintenance vehicle access. The fauna corridor has one metre of fill retained by 300-millimetre-thick reinforced concrete walls supporting a steel wire fence and a perforated aluminium fauna screen on the western side.

Figure 2: Location of the Yalinda Drive Bridge
Click on the image to open Google Maps

The bridge has been designed to be adaptable and resilient for the future and can be converted to a full width road bridge if required, minimising lifecycle costs and contributing to a circular economy. This can be achieved by removing the shared path infill, fauna fill and adjacent retaining walls.

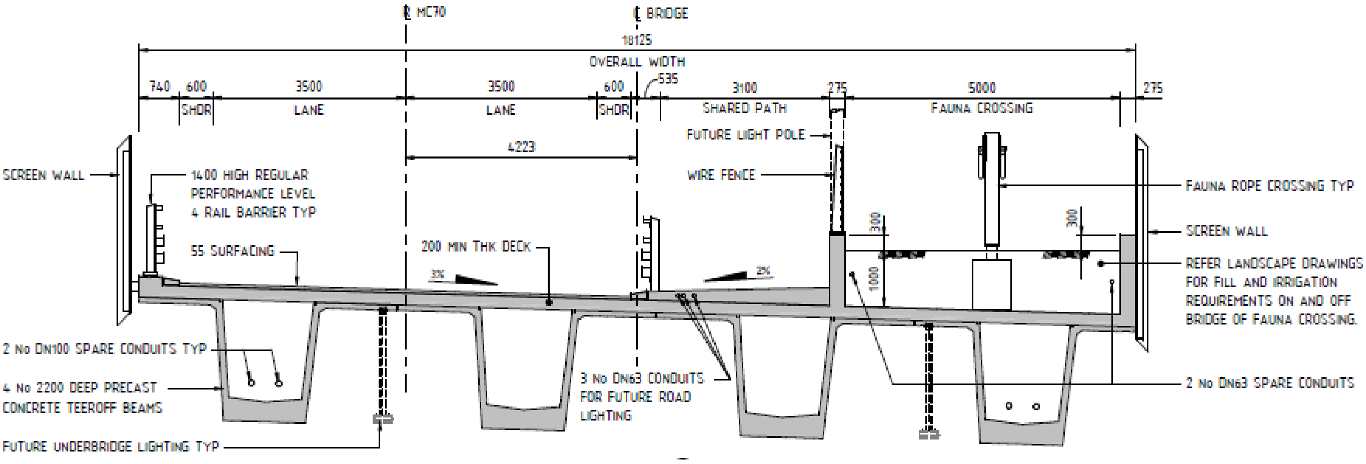


Figure 3: Cross-section of the bridge

FAUNA CROSSING

As part of the conditions for environmental approval, fauna crossings have been designed to maintain habitat connectivity over the road corridor.

The bridge includes one metre of soil fill in the fauna corridor to support vegetation and facilitate fauna movement. This is retained by reinforced concrete walls supporting a steel wire fence and a perforated aluminium fauna screen on the western side.

Landscaping design was a key consideration. The landscaping design promotes connectivity for fauna while strategically placing physical obstructions (such as large logs and boulders) along the bridge to deter unauthorised access by off-road vehicles and motorcycles.

By using natural materials that blend with the surrounding environment, the design remains functional for fauna and reduces the visual impact, while effectively addressing the safety issue.

A landscaping allowance of 2.0 kPa accounts for the type and density of planting (e.g. small, regularly spaced trees) and accommodates heavier landscaping elements such as the logs and boulders.

Drainage and waterproofing were key design considerations to ensure the bridge provides a 100-year design life.

Two fauna rope crossings are supported under the bridge. Additionally, a rope crossing catenary structure is located centrally along the fauna corridor. This is designed to facilitate safe passage for possums while the vegetation along the bridge gets established.

URBAN DESIGN

For the structures on the Wilman Wadandi Highway, the Urban Design Plan specified the architectural requirements for the bridges, including pier form and shape, abutment wing walls shape, integrated artwork for the bridge piers and abutment walls, and colour scheme.

The integrated artwork (relief and paint) on the bridge abutments and wing walls adopts the Noongar six seasons design theme (Figure 6), which also serves as a wayfinding journey along the Highway.

Commencing at the northern entry with the Djilba vibrant sunset theme (refer to Figure 7 for bridge artwork), the alignment travels through Ocean River Sky, Geology Farmland and Rural Bush, finishing at the southern end adopting the Makuru winter sunset theme.

On the Yalinda Drive bridge, the rich colour palette on the abutment walls represents the Noongar season of Djeran (the autumn period from April to May).

Figure 4: Possum rope crossing design

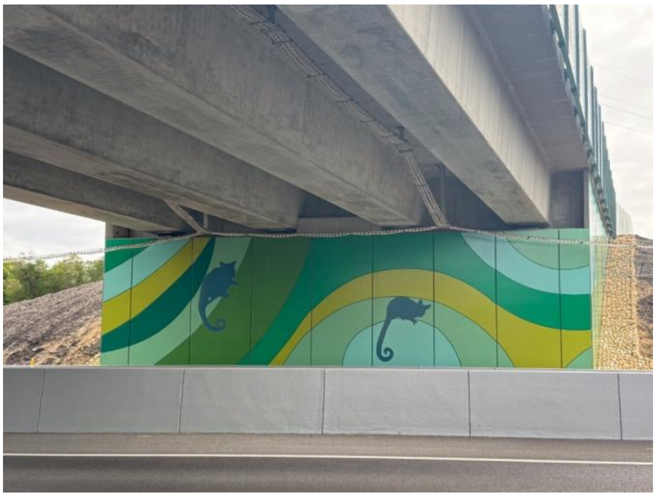


Figure 5: Artwork and fauna rope crossings under the bridge

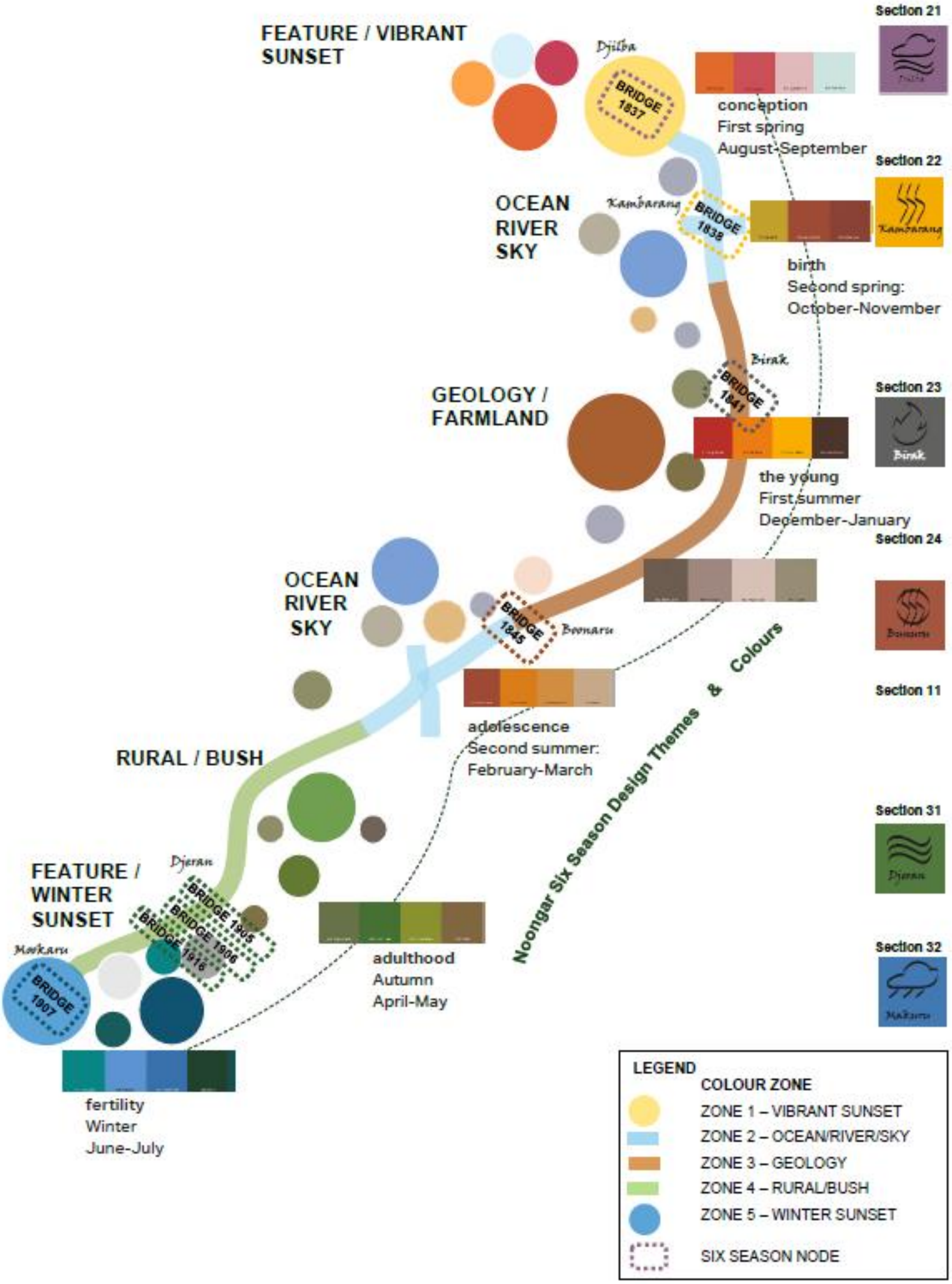


Figure 6: Noongar seasonal theme on the Wilman Wadandi Highway

The artwork, designed by a local Noongar mother-daughter artist team, features the Koomool (Western Ringtail Possum), which are depicted as recessed silhouettes and painted in peacock blue to represent both the nocturnal nature of these creatures and to complete an aesthetic sense of calm for the composition.

The Yalinda Drive bridge demonstrates a legacy and sustainable bridge where vehicles, pedestrians and fauna can all share the space safely and travel efficiently together.

ACKNOWLEDGEMENT

The authors wish to thank the South West Gateway Alliance project team, AECOM and Aurecon (design team), Acciona, NRW Contracting, MACA Civil (construction team) and Main Roads Western Australia for their contributions.

Figure 7: Wilman Wadandi Highway northern entry – Bridge 1837 'Djilba' Bridge

Figure 8: Wilman Wadandi Highway southern entry – Bridge 1907

MANGANUI GORGE SUSPENSION BRIDGE, NEW ZEALAND

Dan Crocker, Chief Bridge Architect and Engineer
DC Structures Studio

Figure 1: Completed Bridge

INTRODUCTION

The Manganui Gorge Suspension Bridge is a long-span suspension footbridge located high on the upper flanks of Mount Taranaki, New Zealand, spanning the Manganui Gorge as part of the Taranaki Crossing Project. The bridge was commissioned by Te Papa Atawhai – the Department of Conservation (DOC) – to remove walkers from a hazardous route through the gorge floor that was subject to rockfall, erosion, and frequent avalanche activity.

Completed in 2024, the project represents a fundamental shift in the way DOC footbridges are conceived, analysed and delivered.

Rather than relying on prescriptive legacy guidance, the bridge was designed using a fully performance-based approach, employing advanced non-linear analysis, modern pedestrian dynamics assessment, and state-of-the-art wind engineering techniques.

The outcome is a structure with a 100-year design life, unrestricted pedestrian capacity and enhanced resilience to extreme alpine hazards, delivered at no additional cost to the client.

Beyond its technical performance, the bridge integrates cultural narratives from Ngāti Ruanui directly into its structural fabric, creating a destination structure that is both functional and deeply connected to place.

BRIDGE LOCATION

The bridge is located close to the summit of Mount Taranaki within Egmont National Park, spanning the Manganui Gorge below the ski field access route. The site is remote, highly exposed and environmentally sensitive, characterised by steep volcanic landforms, complex wind patterns and seasonal snow and avalanche loading.

In addition to its environmental challenges, the location holds significant cultural importance for Ngāti Ruanui and other iwi of Taranaki. This context demanded a solution that was not only robust and low maintenance, but one that respected the maunga and could safely accommodate increasing visitor numbers.

DESIGN BRIEF AND DEVELOPMENT

DOC's initial brief anticipated a conventional suspension bridge designed in accordance with SNZ HB 8630, a standard that has seen limited development over the last two decades. These traditional approaches often result in conservative live loading, limited treatment of dynamic effects and reduced confidence in fatigue and cable-loss behaviour.

Figure 2: Location of the Bridge on the map
Click on the image to open Google Maps

DC Structures Studio proposed a performance-based alternative that combined international best-practice guidance from Eurocodes, PTI recommendations, and specialist pedestrian vibration literature. Sophisticated 3D geometric and non-linear modelling was used from the outset to

KEY FACTS

Location	Mt Taranaki, NZ
Client	Department of Conservation
Cost	$1.3M
Bridge Engineer	DC Structures Studio
Architect	DC Structures Studio
Geotechnical Engineer	Riley Consultants
Main Contractor	Abseil Access
Main Fabricator	PACE Engineering
Cables	Shaws + FATZER
Commencement of design	January 2022
Opening of bridge	April 2024
Type of bridge	Steel Suspension Footbridge
Main span	90m
Usable Bridge Width	1.2m
Vertical clearance	49 m above gorge floor
Design Wind Speed	≈200 km/h

understand the true behaviour of the structure under wind, seismic, pedestrian and abnormal load cases. This methodology allowed live loading to be optimised and structural stiffness to be carefully calibrated, resulting in an unrestricted bridge with improved comfort, reduced material demand and a doubling of the design life from 50 to 100 years.

SUBSTRUCTURE AND FOUNDATIONS

Substructure and Foundations Design

The foundations for the Manganui Gorge Suspension Bridge were governed by a combination of extreme topography, environmental sensitivity, alpine hazards, and the need for long-term durability with minimal ongoing maintenance.

The gorge is steep-sided and deeply incised, with an active avalanche path and highly variable volcanic geology, which ruled out any form of foundation construction on the gorge floor itself.

To address these constraints, the bridge was conceived as a true long-span structure, with all primary foundations located on stable ground above the gorge margins. This approach removed workers, temporary works, and permanent structures from the most hazardous parts of the site, while also significantly reducing environmental impact.

The main bridge system is supported by reinforced concrete anchor blocks and mast foundations, each anchored directly into competent volcanic rock using drilled and grouted rock anchors. These anchors provide the primary resistance to vertical, longitudinal, and uplift forces generated by the main cables, backstays, wind stays, and extreme load combinations.

Particular attention was given to the anchorage demands arising from wind, seismic, avalanche surcharge, and abnormal load cases such as cable loss. Load combinations frequently involved concurrent actions, for example, high wind coincident with snow loading or seismic effects. The foundation system was therefore designed to remain elastic under service conditions and robust under ultimate and abnormal limit states.

The asymmetric geometry of the bridge — driven by the steep rock face on one side of the gorge and more gently sloping terrain on the other — resulted in differing foundation configurations at each end of the span.

Anchor locations and orientations were optimised to align with force resultants from the main cables and backstays, improving efficiency and reducing the overall anchor count.

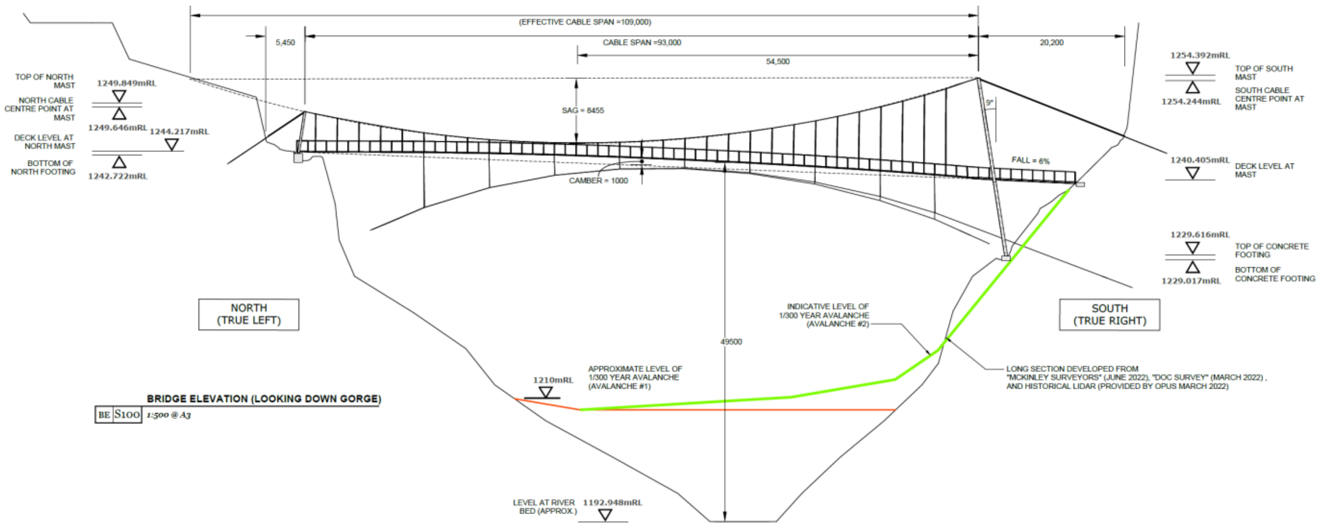


Figure 3: Elevation of the Bridge
Click on the image to open it in a higher resolution

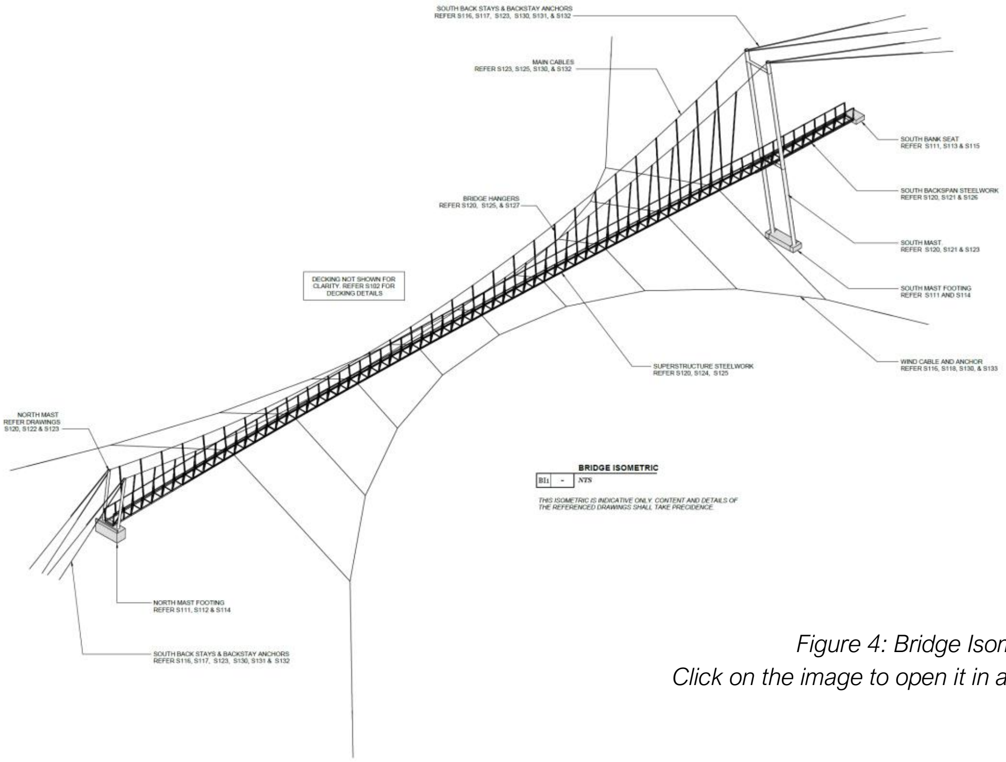


Figure 4: Bridge Isometric
Click on the image to open it in a higher resolution

Durability was a key driver in material selection and detailing. Concrete cover, reinforcement detailing, and anchor corrosion protection systems were all specified to achieve the required 100-year design life in an exposed alpine environment, with freeze–thaw action, moisture ingress, and limited access for maintenance taken into account.

Substructure and Foundation Construction

Construction of the foundations was planned to minimise both environmental disturbance and time spent on site in a highly exposed location. All drilling and concreting operations were undertaken from the gorge margins, avoiding the need for access tracks, scaffolding, or temporary works within the valley floor.

Rock anchors were installed using helicopter delivered drilling rigs capable of operating on steep terrain and constrained working platforms. Drilling operations were carefully sequenced to ensure high-quality bond development within variable volcanic strata. Anchors were proof tested and pre-tensioned prior to incorporation into the permanent works, providing confidence in both capacity and load–displacement performance.

Concrete foundation elements were cast in situ and detailed to accommodate complex anchor geometries, including inclined anchors resisting uplift and longitudinal actions.

Anchor heads and associated steelwork were set out with tight tolerances to ensure accurate alignment with the superstructure geometry and to facilitate efficient cable installation and stressing.

Given the remote alpine setting, foundation construction was closely coordinated with superstructure fabrication and erection. This allowed anchor blocks and mast foundations to be completed and cured in advance of cable installation, reducing the duration of critical lift and stressing operations.

The completed foundation system provides a robust and unobtrusive base for the bridge, largely hidden within the surrounding landscape, while delivering the strength, stiffness, and durability required to support a long-span suspension bridge in one of New Zealand's most challenging alpine environments.

MAST

Mast Design

The mast system forms the primary vertical load path and a defining visual element of the Manganui Gorge Suspension Bridge.

Its design was governed by a combination of extreme environmental loading, asymmetric site constraints, and the need for long-term durability in an exposed alpine setting.

Figure 5: Mast base construction at various phases of construction

Figure 6: Helicopter delivery of the anchoring equipment, anchoring, and final outcome

Unlike many traditional suspension footbridges where towers are arranged symmetrically about the span, the geometry of the Manganui Gorge site necessitated an asymmetric mast arrangement. One side of the gorge terminates in a near-vertical rock face, while the opposing side slopes more gently. The mast locations were therefore offset to suit the available ground conditions and to optimise force flow from the main cables and backstays into the foundations.

The masts are formed from circular hollow steel sections, selected for their structural efficiency, clean aesthetics, and favourable aerodynamic behaviour. The inclined mast geometry allows axial forces from the main cables and backstays to be resolved predominantly as compression, minimising bending demands and improving overall efficiency under both gravity and extreme load combinations.

Structural behaviour of the mast system was assessed using fully non-linear three-dimensional finite element modelling. This approach captured the interaction between mast flexibility, cable pretension, and catenary effects in the suspension system. Particular attention was given to global buckling behaviour, recognising that the mast does not behave as a conventional cantilever. Instead, restraint provided by the stay cables and backstays results in an effective pinned–pinned condition, significantly influencing the governing buckling modes and effective lengths.

Connections at the mast tops, where main cables, backstays, and wind stays converge, represent critical load transfer points. These regions were subjected to detailed finite element analysis to confirm local strength, stiffness, and deformation compatibility under ultimate and abnormal load cases, including scenarios with partial cable loss. Internal stiffening arrangements were developed to ensure that the load was distributed effectively through the mast walls without visually complicating the external form.

Durability was a key consideration in the mast design. All steelwork is protected using a thermally sprayed metal coating system, providing long-term corrosion resistance in an environment characterised by high rainfall, frequent wetting, and freeze–thaw cycling. The detailing philosophy aimed to eliminate water traps and simplify future inspection and maintenance requirements over the 100-year design life.

Mast Construction

Fabrication of the mast components was undertaken off-site in controlled workshop conditions to achieve the tight tolerances required for both structural performance and architectural intent.

Given the convergence of multiple cable systems at the mast heads and the reliance on precise geometry for proper force distribution, fabrication accuracy was critical.

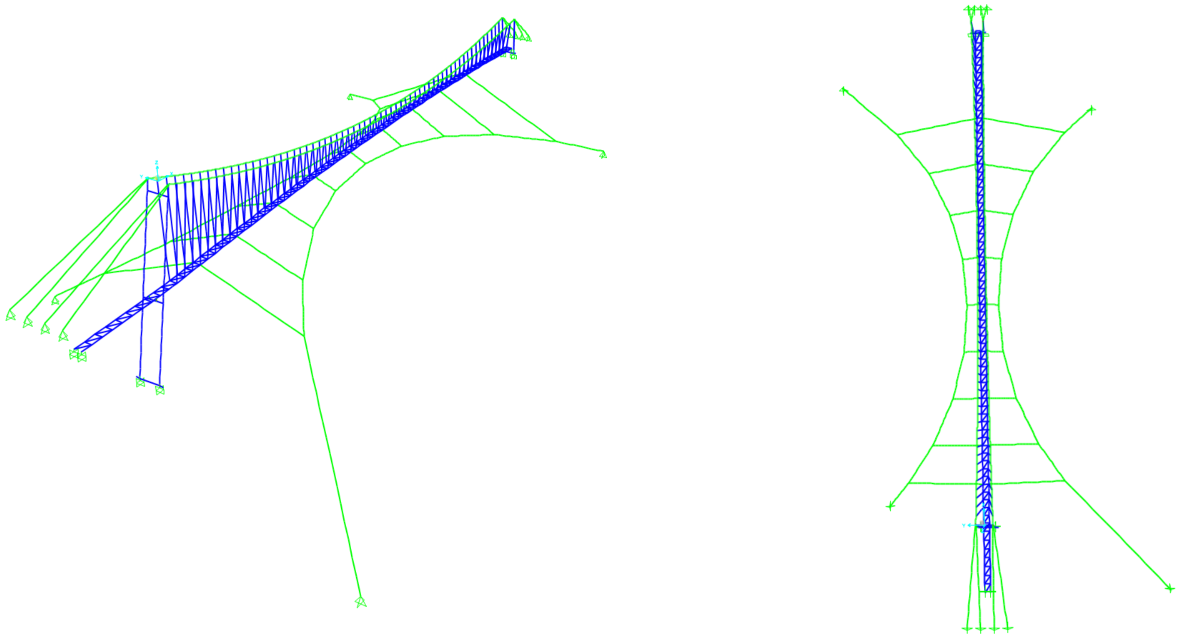


Figure 7: 3D Non-Linear Finite Element Model in SAP2000 v24

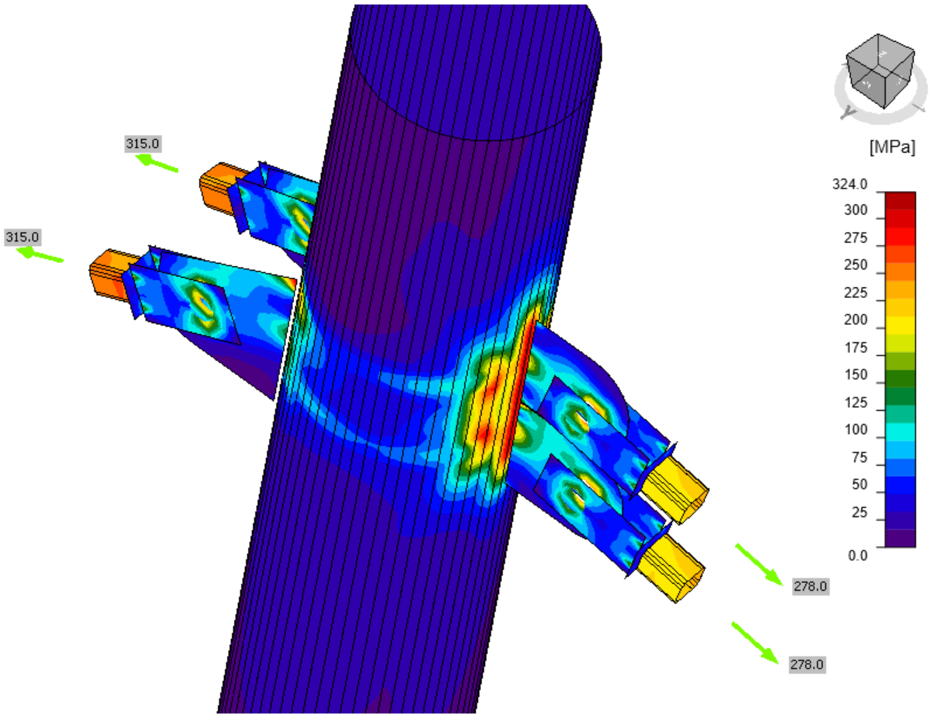


Figure 8: FE modelling of critical socket-to-mast connection

Due to the remote alpine location, steep terrain, and limited ground access at the gorge margins, the mast installation was carried out using heavy-lift helicopter operations. This approach eliminated the need for temporary access tracks, cranage platforms, or large-scale temporary works, significantly reducing environmental disturbance and construction risk in a highly exposed setting.

Mast elements were fabricated as discrete liftable sections sized to suit helicopter lifting capacity and site constraints. Each lift was carefully rehearsed and undertaken within tightly controlled weather windows, with wind limits governing both lifting operations and final placement tolerances. Temporary connection details were designed to allow rapid positioning and secure fixing immediately upon touchdown.

The mast base connections, including anchor bolts and bearing interfaces, were pre-set to tight tolerances to enable direct installation without on-site modification. Once positioned, mast sections were immediately secured to provide stability prior to release of the helicopter load. This methodology minimised time spent working at height and reduced exposure to rapidly changing alpine weather conditions.

Figure 9: Mast components being fabricated at PACE Engineering, New Plymouth

Figure 10: South mast following erection

Following erection, the mast formed the central datum for subsequent installation of the cable systems. Main cables, backstays, and wind stays were installed and progressively stressed, with mast behaviour closely monitored during activation of the structural system. This controlled sequencing ensured that loads were introduced in accordance with the design assumptions and that the final geometry and force distribution matched the analytical model.

The helicopter-based erection strategy proved highly effective for this site, enabling safe, efficient construction while preserving the integrity of the surrounding landscape. Once complete, the mast stands as both a functional structural element and a refined visual feature, its clean form and precise alignment reflecting the care taken during both fabrication and installation.

CABLES

The cable system is the primary load-carrying element of the Manganui Gorge Suspension Bridge and was central to achieving both the long-span performance and the desired structural robustness.

The system comprises main suspension cables, vertical hangers, backstays, and inclined wind stays, all working together as an integrated, prestressed system rather than as isolated components. High-strength spiral strand cables were selected for their proven durability, fatigue resistance, and predictable long-term behaviour in suspension bridge applications.

Cable sizes and layouts were optimised through non-linear three-dimensional analysis, allowing dead load, live load, wind, seismic, avalanche, and abnormal load cases to be considered simultaneously. This approach provided a clear understanding of force redistribution within the cable network under both service and ultimate limit states.

The bridge uses 1x37 1620MPa spiral strand FATZER cables sourced from Switzerland and cut to size and speltered to sockets by Shaw's (Cambridge, NZ). For the main span two 24mm cables per side are adopted, and for the wind cables 20mm are used. Corrosion protection of the cables is achieved by the adoption of a minimum 250g/m^2 coating of 95% zinc + 5% aluminium to all strands making up the strand.

Cables are designed as per the Post Tensioning Institute (PTI) Recommendations for Stay-Cable Design, Installation and Testing, 6th edition 2012.

A defining feature of the bridge is the calibrated prestressing of the wind stays. In contrast to traditional suspension footbridges in NZ, where wind cables are often installed slack and only become effective once significant movement occurs, the wind stays at Manganui Gorge were deliberately prestressed.

Figure 11: Spiral strand stay cable detail (left), back-span cables (right)

This strategy actively stiffens the bridge deck in both vertical and lateral directions under everyday loading, improving pedestrian comfort and aerodynamic performance while also contributing to redundancy under extreme events.

Cable loss scenarios were explicitly considered as part of the design development. Time-history dynamic analyses were undertaken to simulate the sudden loss of individual cables due to rockfall or other accidental actions. These analyses demonstrated that the prestressed cable system redistributes load rapidly, with transient increases in adjacent cable forces remaining within acceptable limits. The system therefore exhibits inherent robustness, preventing progressive collapse and maintaining overall structural stability following localised damage.

SUPERSTRUCTURE

Superstructure Design

The superstructure of the Manganui Gorge Suspension Bridge was conceived as a lightweight, aerodynamically stable system capable of spanning the full width of the gorge without intermediate support. This approach removed the structure from avalanche paths and rockfall zones while allowing all critical components to be located on stable ground beyond the gorge edges.

The bridge consists of a single-span suspended walkway supported by main cables, vertical hangers, and a system of inclined prestressed wind stays. Unlike traditional DOC suspension bridges, where wind cables often remain slack until engaged by large movements, the Manganui Gorge bridge adopts a deliberately calibrated prestressing strategy. This approach actively stiffens the structure under service conditions, improving both aerodynamic and pedestrian performance.

The deck system is formed using a porous fibre-reinforced polymer (FRP) decking supported on longitudinal edge members and transverse floor beams.

The choice of FRP was driven by several factors: low self-weight, high durability in an alpine environment, resistance to freeze–thaw degradation, and favourable aerodynamic behaviour. The porous nature of the deck allows air to pass through the walkway, significantly reducing uplift pressures and vortex formation beneath the deck in high wind conditions.

Deck geometry, hanger spacing, and cable profiles were all developed using a fully non-linear 3D analysis model. This allowed the final bridge profile, camber, and cable forces to be precisely defined in advance, minimising on-site adjustment and ensuring that the completed bridge achieved its intended geometry and performance characteristics immediately following construction.

Balustrades form an integral part of the superstructure both structurally and architecturally.

Stainless steel infill cables provide pedestrian containment while remaining visually light, preserving views across the gorge. Discrete laser-cut stainless steel panels are incorporated at defined locations and act as both functional safety elements and cultural artworks, seamlessly integrated into the structural system rather than applied as surface decoration.

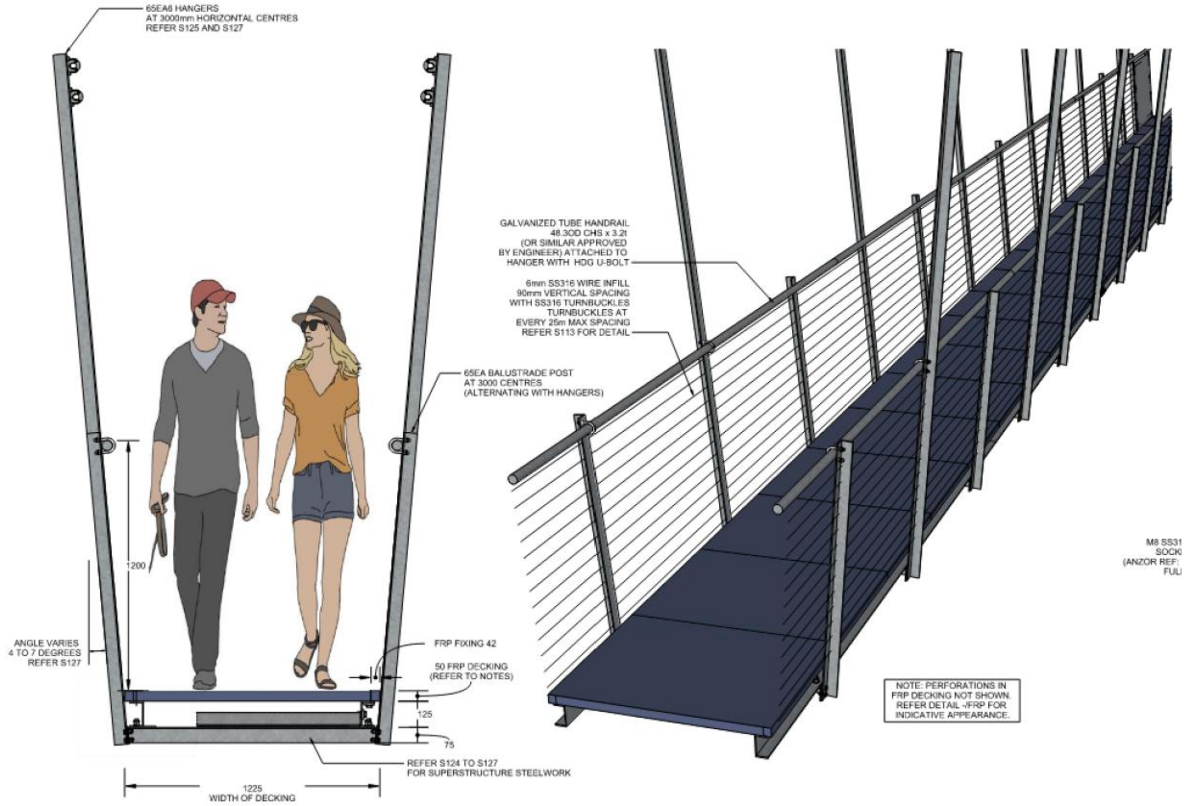
Superstructure Fabrication and Erection

Given the remote alpine location and exposure of the site, fabrication and erection of the superstructure were heavily influenced by constructability and safety considerations. All primary steel components were fabricated off-site under controlled conditions to ensure quality, dimensional accuracy, and protection of critical finishes.

Superstructure erection was planned to minimise the duration of high-risk activities at height and in exposed weather conditions. Main cables were installed and stressed first, followed by the controlled installation of hangers and secondary cable systems. This staged approach allowed the structural behaviour of the bridge to be closely monitored at each step and ensured that load was introduced to the system in a predictable and well-understood manner.

↑ *Figure 12: Hangers being installed*

↓ *Figure 13: Bridge cross section details*



The deck modules were designed to be lightweight and manageable within the constraints of the site's access and lifting equipment. Installation was carried out incrementally, with deck units attached to the hangers in a balanced sequence to control movements and cable forces. Because the final geometry and force distribution were fully established in the design model, the need for on-site adjustment was minimal.

Prestressing of the wind stays was undertaken as a deliberate and controlled operation, forming a critical step in activating the final stiffness of the system. This process ensured that the bridge achieved the target natural frequencies and serviceability performance immediately upon completion, rather than relying on passive engagement under extreme loading.

The completed superstructure presents a visually simple and elegant form, with structural elements working efficiently and clearly expressing their purpose. The purity of the suspended deck, combined with the subtle integration of cables, balustrades, and artwork, results in a bridge that is both technically refined and highly sympathetic to its dramatic alpine setting.

INTEGRATION OF ART AND CULTURAL NARRATIVES

Ngāti Ruanui are a Māori iwi (tribe) of the Taranaki region of Aotearoa New Zealand, with deep ancestral, cultural, and spiritual connections to Mount Taranaki and the surrounding landscape.

A genuine partnership with Ngāti Ruanui was established from the earliest stages of the project, moving beyond consultation to collaborative design. Iwi aspirations centred on creating a bridge that was safe, enduring and expressive of cultural narratives associated with the maunga and the Manganui Gorge.

Figure 14: Completion of superstructure

Figure 15: Architectural laser-cut balustrade infill panels

Cultural artwork was integrated directly into the structural systems rather than applied as surface decoration.

Laser-cut stainless steel balustrade panels serve both as safety elements and as cultural canvases, depicting the phases of avalanche cycles that are central to local kōrero.

Sculptural stainless-steel elements at the masts were developed collaboratively with Ngāti Ruanui artist Wharehoka Smith and engineered using finite element analysis to ensure structural integrity under extreme loading. By embedding culture into functional components, the project achieved meaningful cultural expression without material or cost penalties.

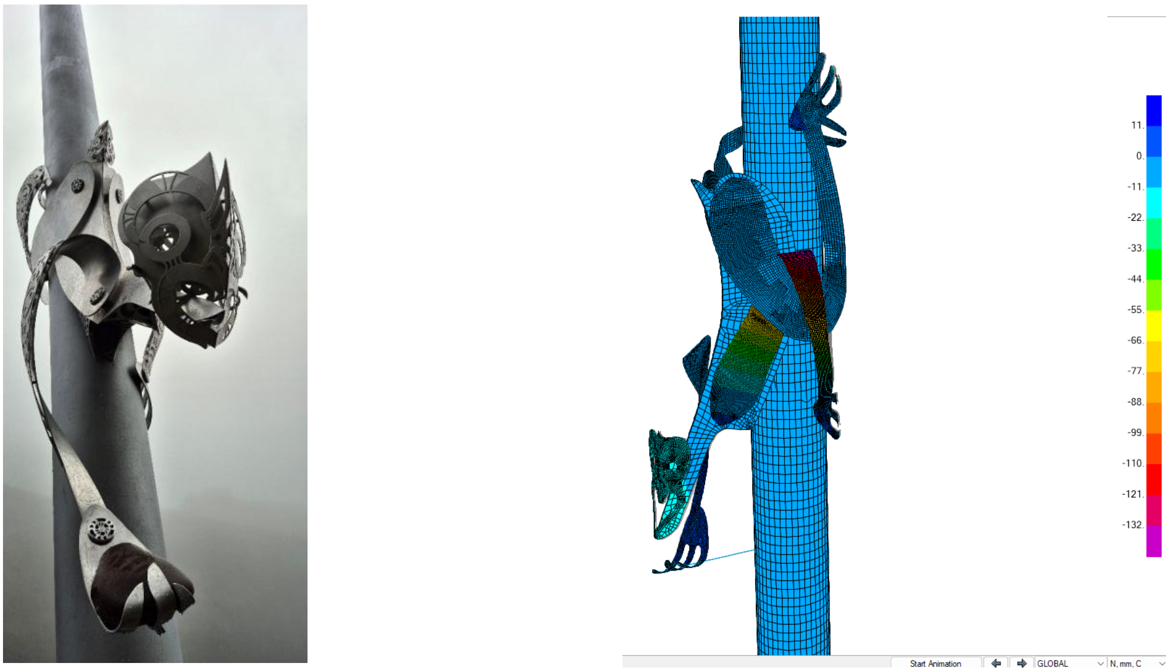


Figure 16: Toanga (treasure) sculpture attached to mast to welcome visitors onto bridge (left); FEA model to confirm the sculpture performance in high winds (right)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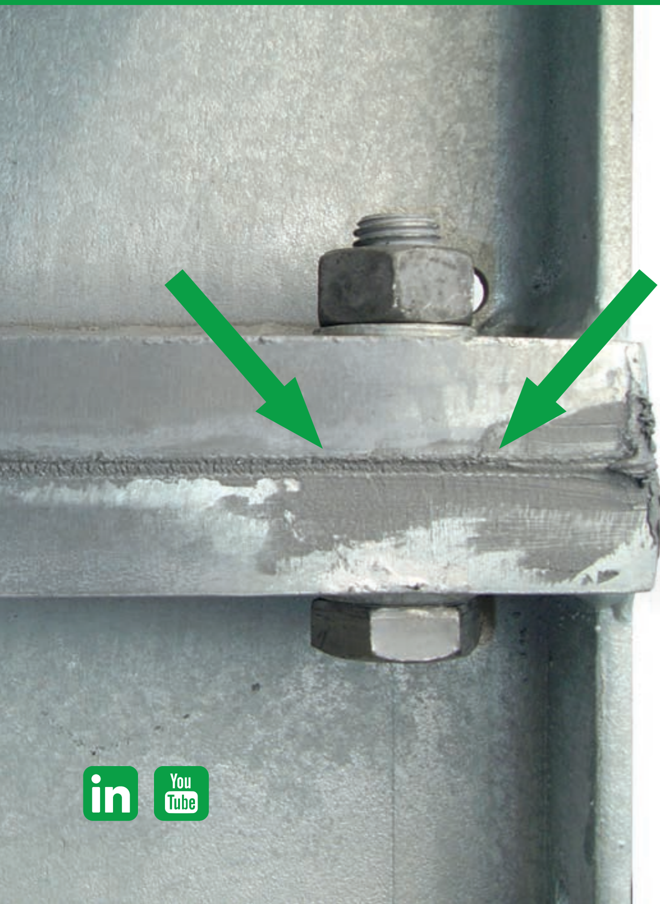
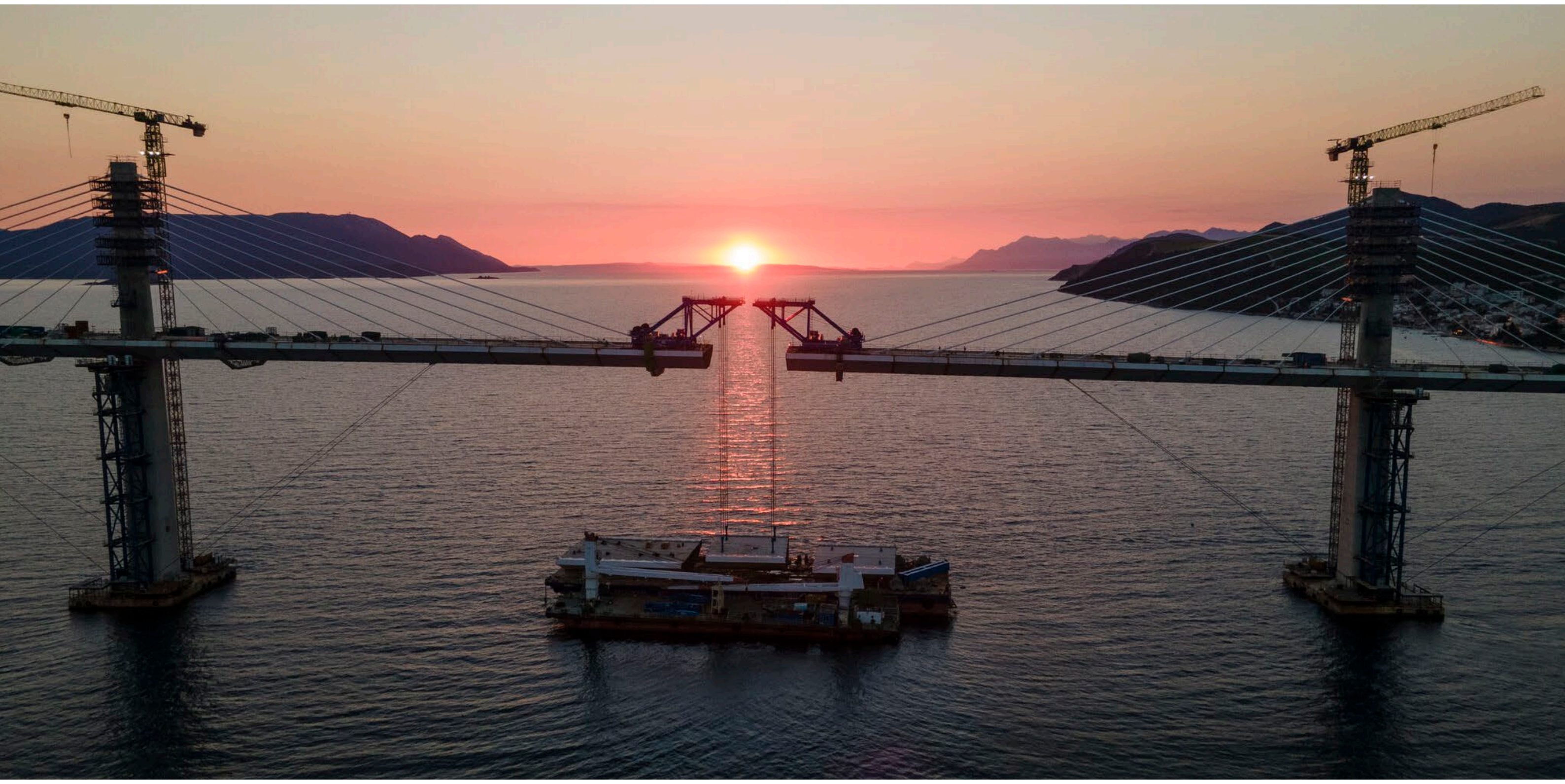
Pedestrian Bridges


Cable-Stayed Bridges




MM1018
Putty - Component A
#1436A
MM1018
Putty - Component B
#1436B
GERMAN GENERAL BUILDING
AbZ
APPROVAL



Pelješac Bridge, Croatia

Conceptual/Preliminary/Final design
Joint Venture Faculty of Civil Engineering, University of Zagreb; Ponting; Pipenbaher Consulting Engineers

Ada Bridge over Sava in Belgrade, Serbia

Winning competition design/Preliminary/ICE for final and detailed design


CCLEX MPTC
acciona
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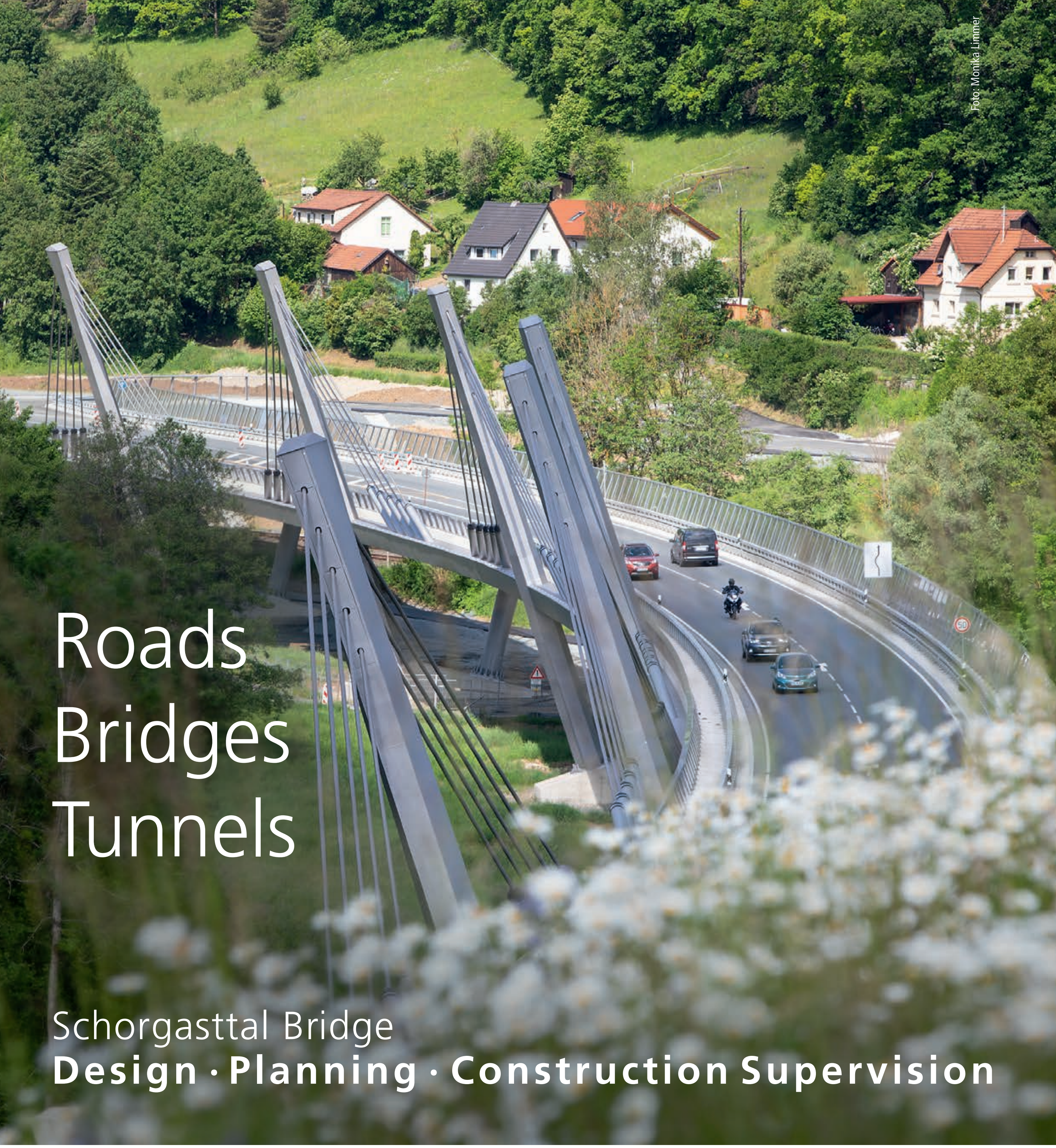
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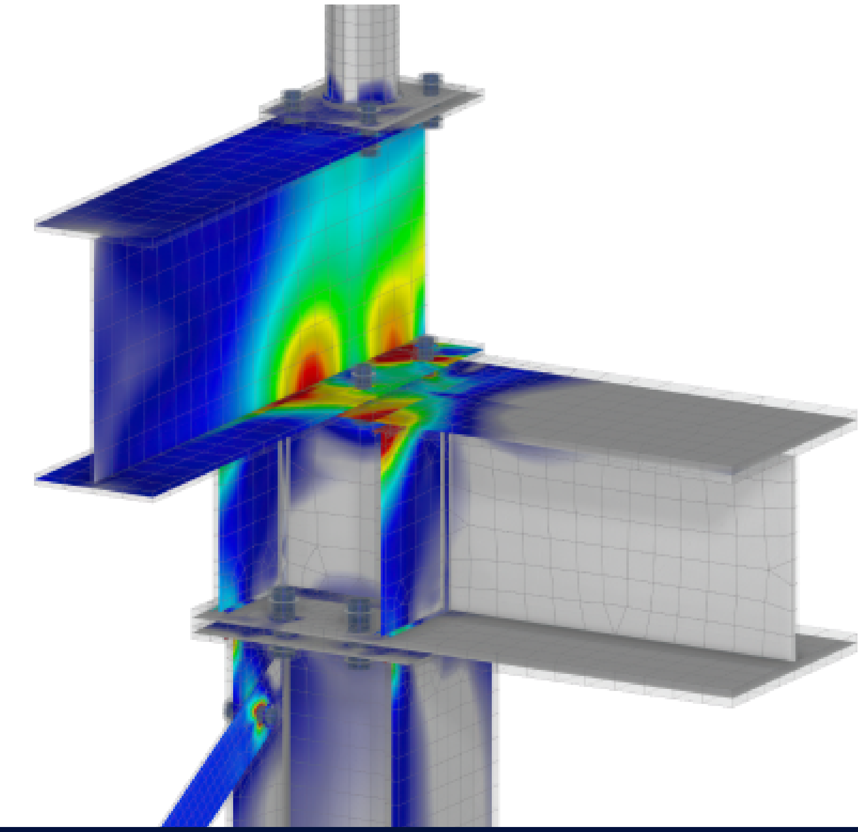

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San Francisco-Oakland Bay Bridge East Span

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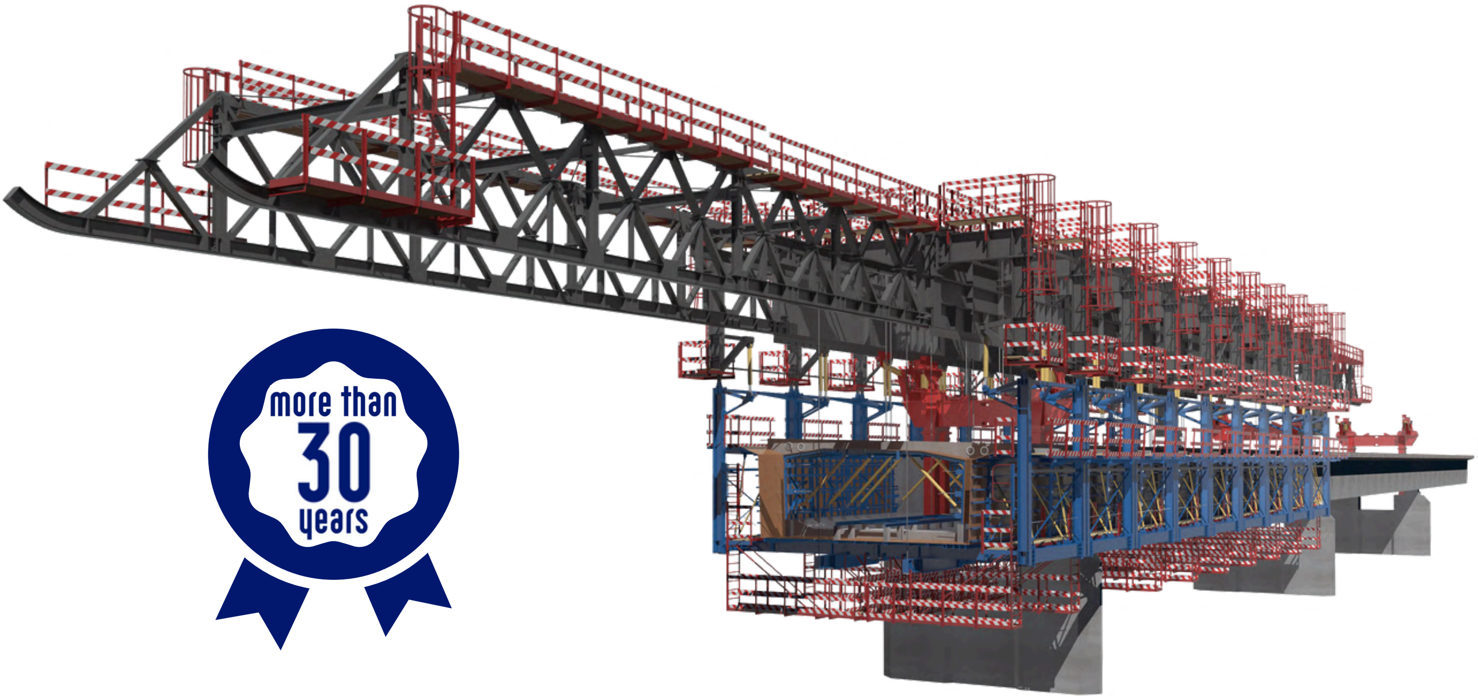
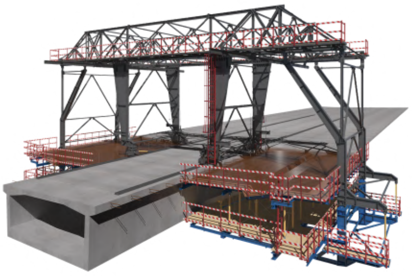
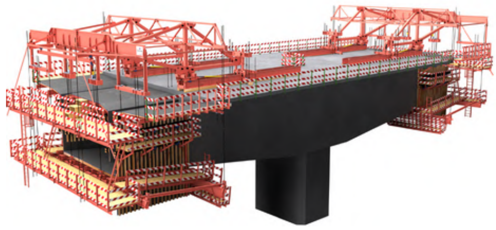






Gerald Desmond Bridge © Marie Tagudena

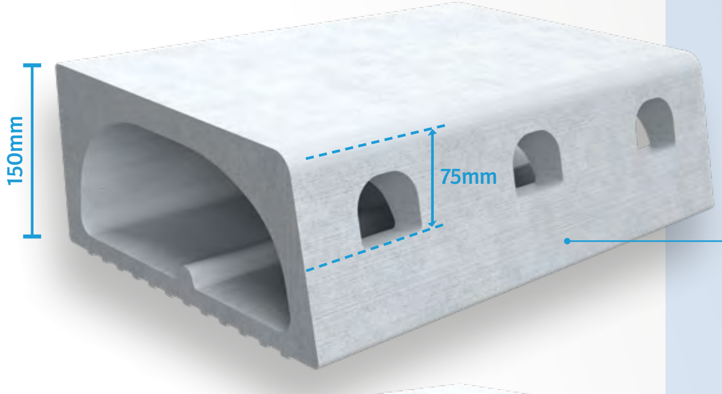
Suitable for 75 and 150 mm Kerb height and all heights in between

BRIDGE DRAINAGE UNIT

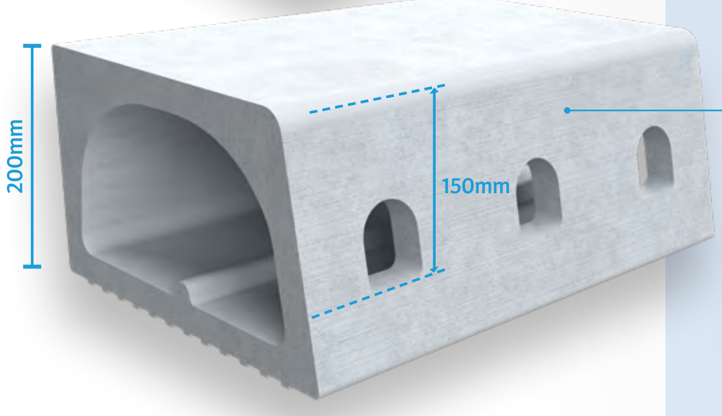
The bridge drainage channel Type M is designed to collect and discharge surface and structural water from bridges and elevated roads, used by all types of road vehicles.

AVAILABLE HEIGHT OPTIONS

The height of the inlet openings is milled to project-specific dimensions.



BD350x150 = 150mm high element
Top slope 4%
(according to BAST guideline Kap12)



BD350x200 = 200mm high element
Top slope 2%
(according to BAST guideline Kap12)

TECHNICAL FEATURES AND ADVANTAGES

The bridge drain elements are made of one material and are moulded monolithically. In other words, they are manufactured in one piece, which ensures a stable structure with high impact resistance.

CUSTOM SOLUTIONS

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- Protection of the bridge deck from horizontal overload caused by extreme closing movements during the earthquake
- High life time expectation through use of high performance components
- Longitudinal seismic displacement of ca. 4 m
- Service velocity up to 20 mm/sec (10 times higher than for a regular bridge)
- Watertight across the bridge width
- Maintenance free

References:
- Bahia de Cadiz, Spain
- Hochmoselübergang, Germany
- Osman Gazi Bridge, Izmit, Turkey
- Mainbrücke Randersacker, Germany
- Millau Viaduct, France
- Rheinbrücke Schierstein, Germany
- Rion Antirion, Greece
- Russky Island Bridge, Vladivostok, Russia
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Attract Top Talent

Join a team of volunteers to build a life-changing footbridge or water project alongside an isolated community as part of our **Free Agent Program**. Or bring a whole team from your company as part of our **Industry Program.** More interested in recruiting? **Become a mentor** for students in our **University Program** to pass on your skills while attracting top talent from over two dozen elite universities around the world. Or donate **in-kind** tools, services such as engineering or project management, or materials like wire rope.

We're breaking the barriers that perpetuate poverty by building essential infrastructure, and we need your help. Contact us today to learn how you or your company can get involved.

Think about it: you, us, a bridge.

Partnership Opportunities Binder

PARTNER WITH US TO BUILD BRIDGES AND WASH SYSTEMS THAT CONNECT PEOPLE TO HEALTH, EDUCATION, AND OPPORTUNITY.

ENGINEERSINACTION.ORG

Fika

We envision safe arrivals for everyone, everywhere.

A world where over one billion currently underserved rural people are connected to essential services and opportunities.

Our mission is to enable safe, reliable rural transport access by helping governments and local actors independently plan, fund, design, build, and maintain the infrastructure their communities need.

Partners help us turn this vision into reality by supporting rural infrastructure that transforms lives and strengthens economies.

Together, we're building the systems that make safe access inevitable.

BRIDGING THE GAP AFRICA

BUILDING BRIDGES & TRANSFORMING LIVES

THE NEED IS GREAT. THE NEED IS NOW.

BE A PART OF THIS. BE A BRIDGE.

e-mosty

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