Kangaroo Point Green Bridge over the Brisbane River, Brisbane (Australia)

Pasarela Kangaroo Point en Brisbane (Australia)

Andrew Gallagher[a], Daniel Nelles[b], Gerhard Du Plessis[c], José Manuel Martínez [d]

[a] Civil Engineer. WSP Australia. Technical Director. andrew.gallagher@wsp.com

[b] Civil Engineer. WSP Australia. Principal Structural Engineer. daniel.nelles@wsp.com

[c] Civil Engineer. WSP Australia. Technical Executive. gerhard@cablestayedbridge.com

[d] PhD Civil Engineer. WSP Spain - APIA. Bridges & Civil Structures Manager. josemanuel.martinezgarcia@wsp.com

ABSTRACT

The Kangaroo Point Green Bridge in Brisbane is a landmark pedestrian and bikeway bridge that connects the central business district to Kangaroo Point. Featuring a single sculptured pylon and a cable-stayed main span, the bridge minimizes the number of piers in the Brisbane River, enhancing flood performance. Spanning a total of 460m, it includes a 183m main span and a 90m asymmetrical back span. This paper discusses the bridge's structural form, including its foundations, piers, pylon, and steel composite deck. It also covers the structural analyses performed, the erection process, and the advanced dynamic analyses related to wind effects and vibrations.

RESUMEN

La pasarela de Kangaroo Point en Brisbane es un puente emblemático para peatones y ciclistas que conecta el distrito central de negocios con Kangaroo Point. Con un único pilón arquitectónico y un vano principal atirantado, el puente minimiza las pilas en el río Brisbane, mejorando el comportamiento ante crecidas. Con una longitud total de 460m, incluye un vano principal de 183m y un vano trasero asimétrico de 90m. El artículo describe la forma estructural del puente, incluyendo cimientos, pilas, pilón y tablero mixto, abordando los análisis estructurales realizados, con mención al proceso constructivo y a los análisis avanzados de tipo dinámico relativos a efectos aerolásticos y vibraciones.

KEYWORDS: Footbridge, cable-stayed, iconic, steel, green bridge.
PALABRAS CLAVE: Pasarela, atirantada, icónica, metálica, sostenible.

1. Project Background

The newly opened Kangaroo Point Green Bridge is a new pedestrian and bicycle bridge connecting Brisbane's Central Business District to Kangaroo Point. Key features include a 183m cable-stayed main span supported by a tall mast with an asymmetrical back span, a solution that minimizes hydraulic effects in a flood-sensitive location, provides large navigational clearances for a busy waterway, and creates a recognizable structure for this vibrant, growing city. The bridge's approach spans have curved geometry to suit landing connectivity requirements. Architecturally profiled piers support the bridge, with a mid-river pier featuring a cradle arrangement for the steel mast structure. The mast includes maintenance access, architectural facades, and integrated lighting. The shared path deck is a 6.8m wide steel composite box section with a 1.45m depth, providing a 3.8m vertical clearance for maintenance vehicles. Utilities are housed within the box section, accessible through maintenance hatches from deck surface.

Figure 1. Artist Impression. Copyright Owner: Brisbane City Council. Credit: Connect Brisbane.

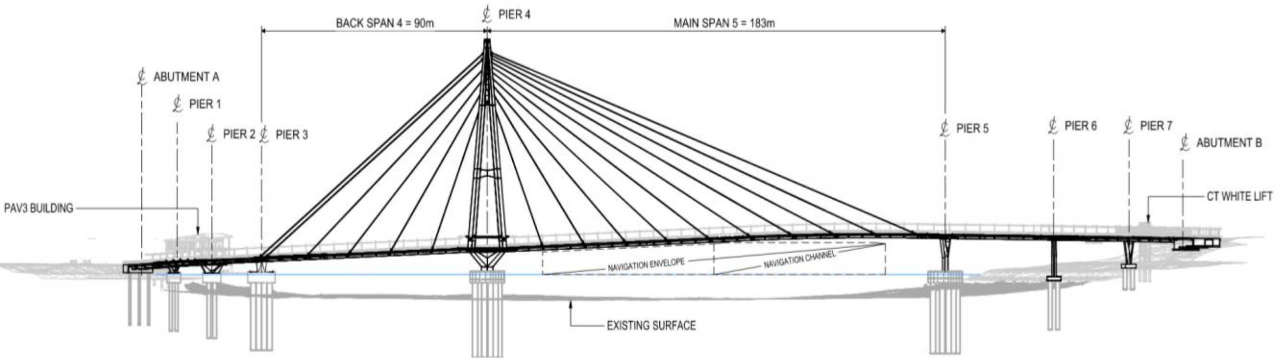


Figure 2. Bridge Arrangement.

Unique "pause points" are located at the mid-river pier, main span, and approach spans, allowing the public to enjoy panoramic views of Brisbane. The bridge segregates bikeway and pedestrian walkways, with shade and rain protection for pedestrians and a raised balustrade for cyclists. Commercial activation includes a building with a restaurant and function rooms at the City Landing approach.

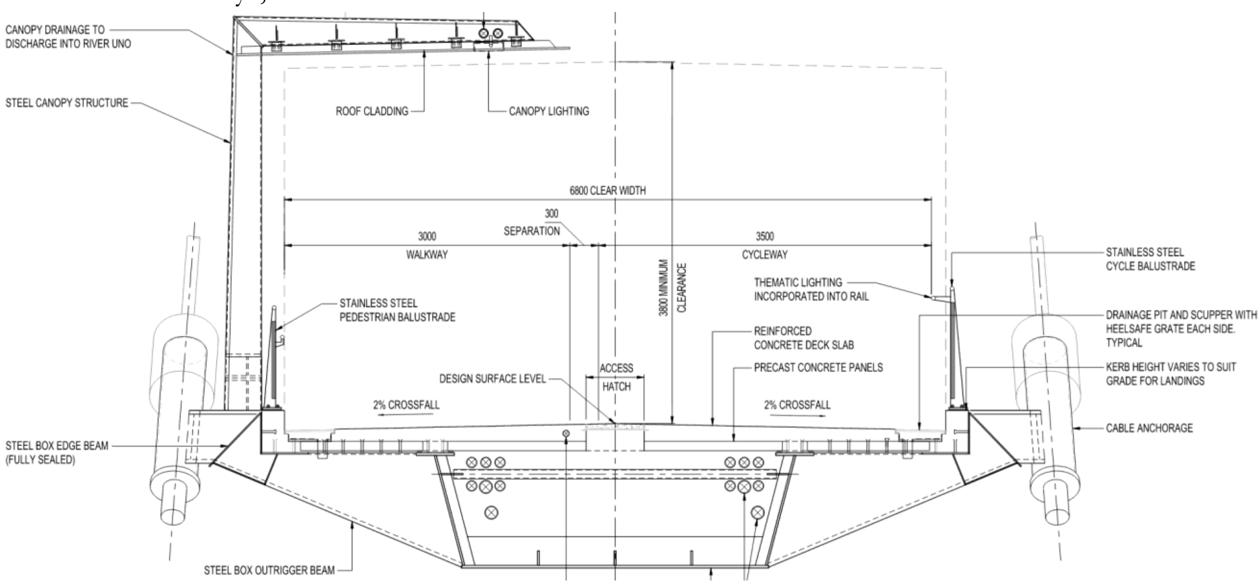


Figure 3. Typical Main Span Section.

2. Foundation Design

The Kangaroo Point Bridge crosses various geological formations, including Neranleigh Fernvale beds and Brisbane Tuff, where pier and abutment foundations are set. A pier scour assessment followed US guidelines, optimizing pile group geometries. River pier piles are cast in place reinforced concrete within a sacrificial driven steel casing, 2.1m in diameter, with length reaching 26m. Foundation design focused on vessel impact resistance, with major piers designed to withstand a 1500-tonne barge. Pier 3 required a "tie down" pier with a different shape that can provide a more direct load path to the large diameter piles, with concrete ballast, and measures to limit uplift force in stay cables.

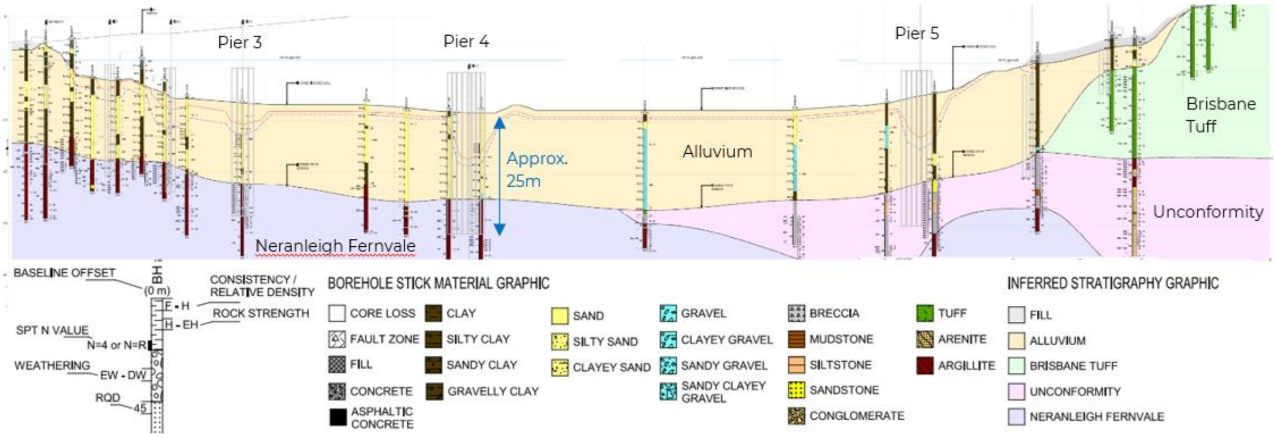


Figure 4. Geological Longitudinal Section.

The constructability of the river pile caps was improved using a precast pile cap shell and skirt arrangement. The shell was transported by barge and positioned over the piles above low tide level. For single-row pile piers (Piers 1, 2, 3, 6), the shells were precast as single units, while for larger piers (Piers 4 and 5), they were made in three units, assembled, sealed, and dewatered. Precast concrete skirts were placed around the perimeter and secured with bolts. A first stage concrete pour strengthened the shell base, the remaining pile cap depth was cast in a second stage pour.

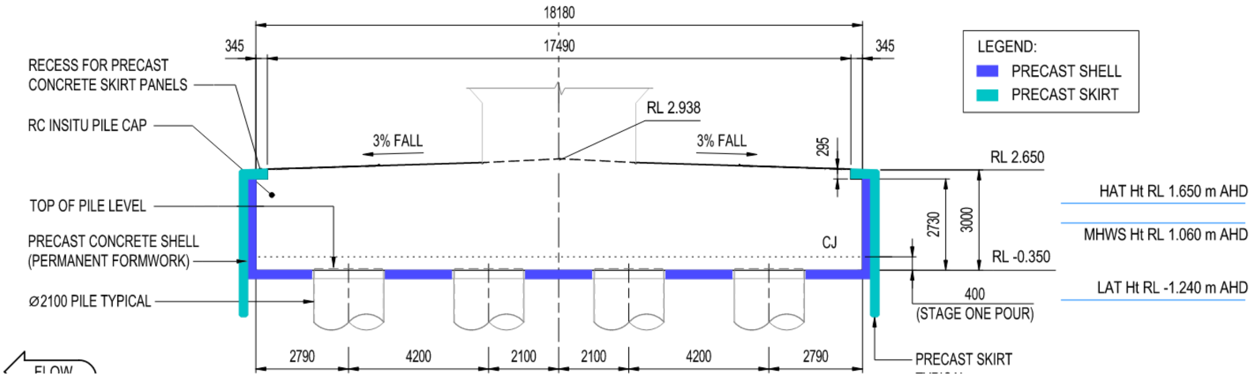


Figure 5. Typical River Pier Pile Cap Arrangement.

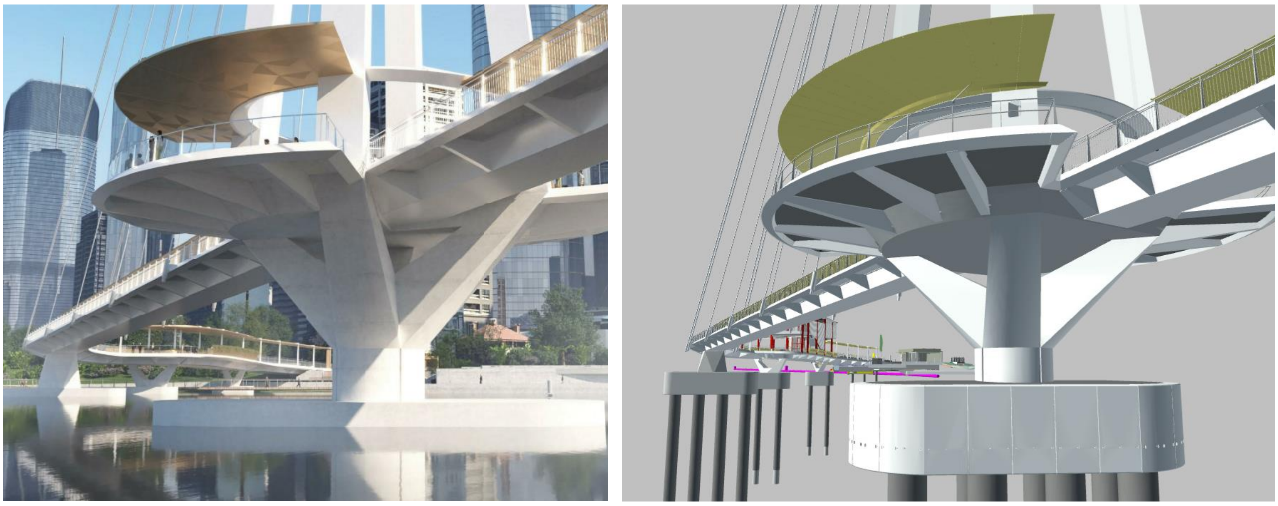
3. Piers

The Project Architects developed a family of pier shapes, giving the bridge its striking sculptured form. The fork-shaped piers at Piers 1, 2, 5, 6, and 7 maintain consistent cross-sectional shapes while allowing variations in height and support width, providing the necessary structural capacity for SLS and ULS flood events and large item impacts. Pier 3, a solid pier with a faceted arrangement, serves as the back span "tie down." This was achieved using plain column reinforcement from the pile cap to the deck diaphragm, preferred over post-tensioned bars due to maintenance challenges.

Photo taken from Kangaroo Point **Detailed Design Digital Structural Model**

Figure 6. Typical Forked Pier Geometry

Pier 4 supports the main mast tower with a cradle form that transitions the four legs of the upper mast to a single 4.6m diameter column. The cradle is laterally tied at deck diaphragm level to limit bending actions in the cradle, with complex detailing for transitions and anchorage of diaphragm tie bars. The deck diaphragm transfers action between spans and supports the cantilevered deck for observation platforms. It also houses bridge utilities. The reinforced concrete diaphragm was cast in multiple stages, with the first stage supporting the steel outriggers before being cast with the second and third stages.



Artist Impression of Pier 4 **Detailed Design Digital Structural Model**

Figure 7. Pier 4 cradle and diaphragm geometry

4. Main Span Superstructure

The main span superstructure features a 1210mm deep steel box girder with a composite concrete deck and internal bracing. Precast deck panels act as formwork for in-situ concrete pour. Steel edge beams and regularly spaced outriggers support the deck cantilever overhang and provide architectural form. At cable stay anchorage locations, outriggers transfer loads to the steel box girder. Widened deck areas for midspan and Pier 5 “pause points” are included in the steel module assembly.

The deck is monolithic at Piers 3 and 4, with sliding guided bearings at Abutment A, city side of Pier 3 half joint, Pier 5, Kangaroo Point side of Pier 6 half joint and Abutment B.

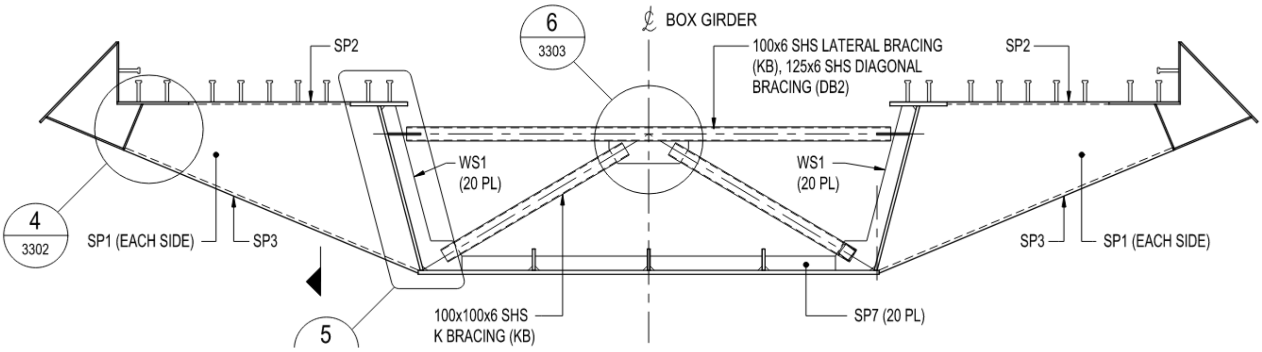


Figure 8. Typical main span deck section.

5. Pier 4 Mast

The mid-river mast structure is the bridge's landmark feature, anchoring the stay cables and standing approximately 93.5m above high tide.

A mid-height cruciform cross bracing enhances the structural performance of the four legs, reducing dimensions and saving steel. The cruciform also has integrated feature lighting accessible from inside the mast. The mast legs, designed as scalene quadrilaterals, taper over the height and are oriented diagonally for improved wind performance. They are fabricated from 20mm plate with internal stiffeners, connected to a 2.8m high concrete plug at the base.

The structure is accessible internally for inspection and maintenance, with an access door at the base and internal stair access in one leg. Lower legs have space for stairs and landings, while upper legs and the mast head have ladders with fall arrest provisions.

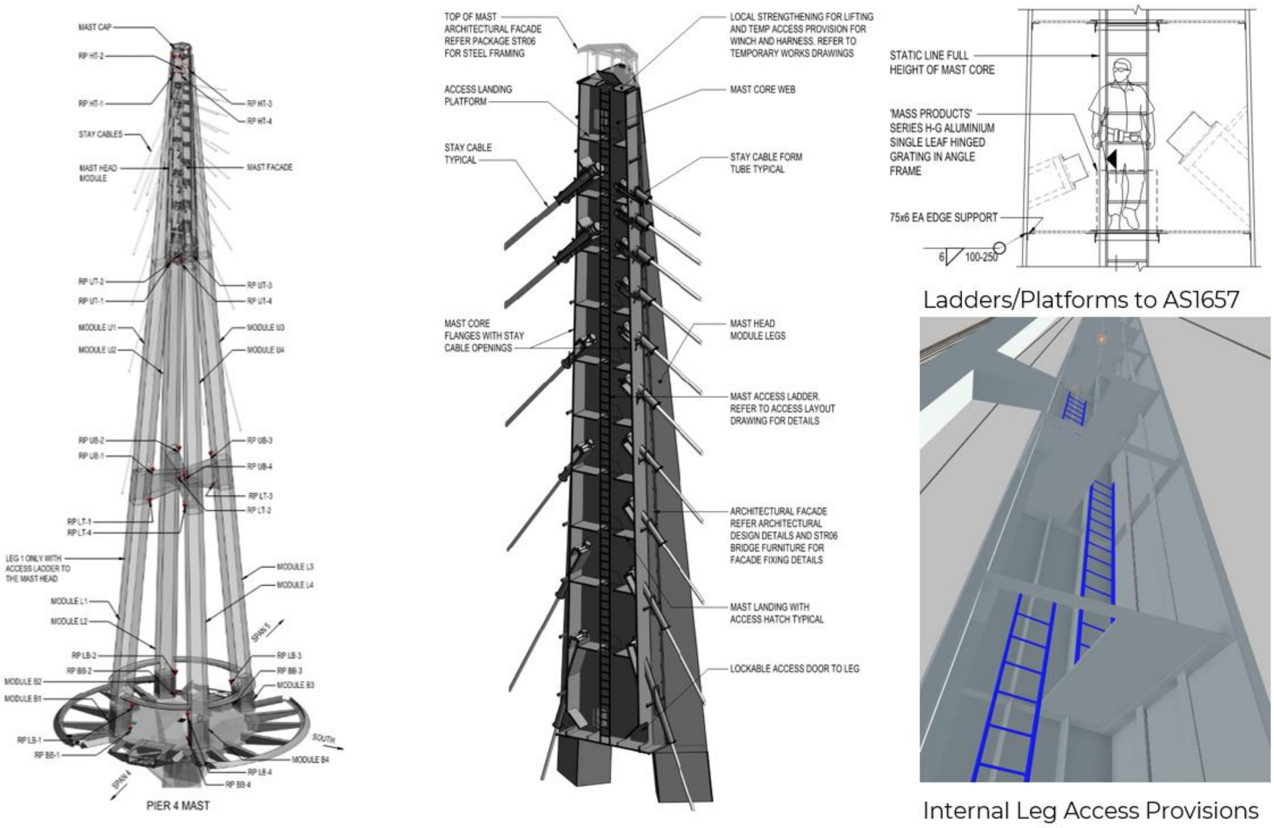
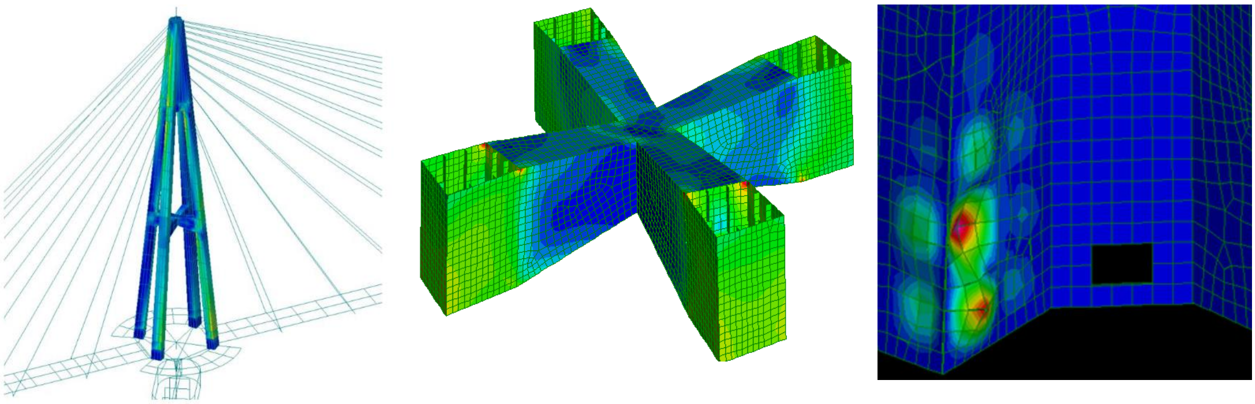


Figure 9. Mast Arrangement and Maintenance Access.

At the top of the mast is the 180-ton mast head structure, which included significant steelwork for the stay cable anchorages and for the single lift. Mast steelwork was checked for loads from the global staged analysis models using finite element models, studying local effects at connection details and stress concentrations at cable anchorages.



Global Model **Example Local Finite Element Model** **Local Plate Buckling Analysis**

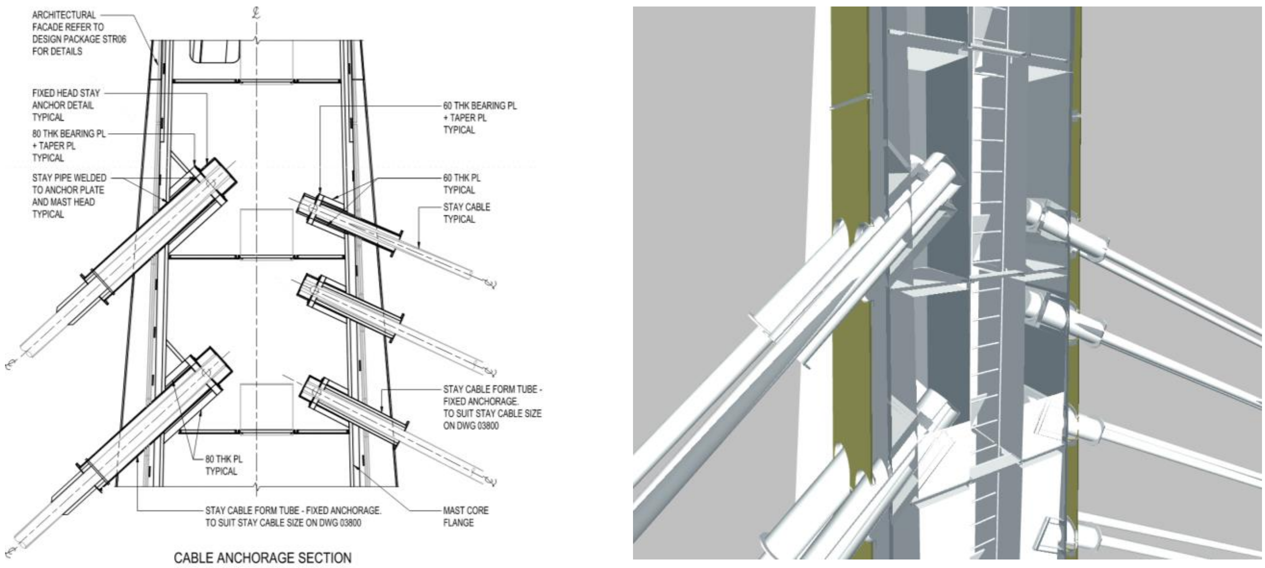
Figure 10. Mast Design Analysis

The mid-height cruciform provides sufficient restraint to prevent global buckling of the mast leg before plastic hinges form. The mast design considered local buckling of the longitudinally stiffened leg panels. Finite element analysis showed satisfactory compression stresses in most external plates. Longitudinal stiffeners were sized to prevent panel buckling, and transverse stiffeners were aligned with landing levels for internal stair access.

Mast horizontal deflection was assessed at the end of construction, considering cable stressing forces and construction sequence. The final deflection was moderate, less than 50mm for the 93.5m high pylon, making pre-cambering unnecessary.

6. Stay Cables

Stay cables were specified as a parallel strand system for individual strand replacement, with typical cables having 12 or 15 strands, and backstay cables having 43 strands. Proprietary cable anchorages with long-term durability protection were used. Cable pipes with double helical fillets improved wind-rain vibration performance and included vandalism protection at deck level. Fatigue testing requirements followed Fib Bulletin 89, with a maximum SLS load target at 50% UTS. Cable loss and replacement combinations were also considered.

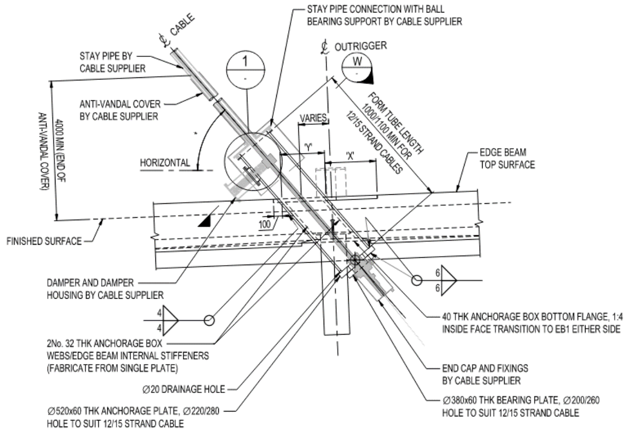


Section through mast head at stay cable anchorages **Detailed Design Digital Structural Model**

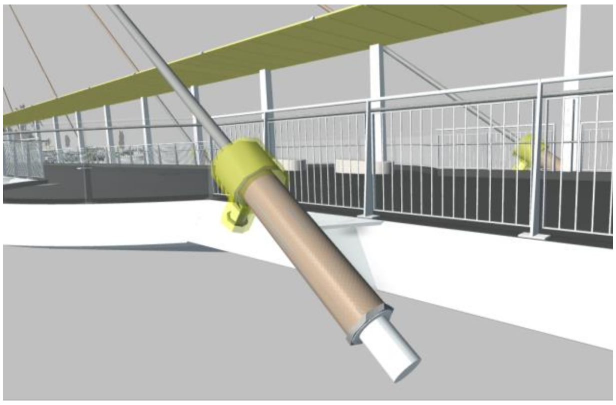
Figure 11. Pylon Mast Stay Cable Anchorages

The cable anchorages are designed to resist 90% of the cable's ultimate tensile strength per Fib Bulletin 89. This ensures overstrength and allows post-yield deformation. Each mast anchorage supports two cable pairs, transferring loads to the mast core steelwork via parallel transfer plates. These plates, sealed for durability, form a box section for load transfer. The largest plate is 80mm thick for 43-strand cables, and 60mm for 12 and 15-strand cables.

At deck level, a heavy gauge form tube transfers anchor forces to the edge beam, provides durability protection, and houses cable vibration damping systems. The proposed damper is an internal viscous piston attached to the form tube and clamped to the cable, with a housing that moves with the cable for maintenance access. The design balances architectural finish with functionality.



Stay Cable with Integrated Cable Damping System

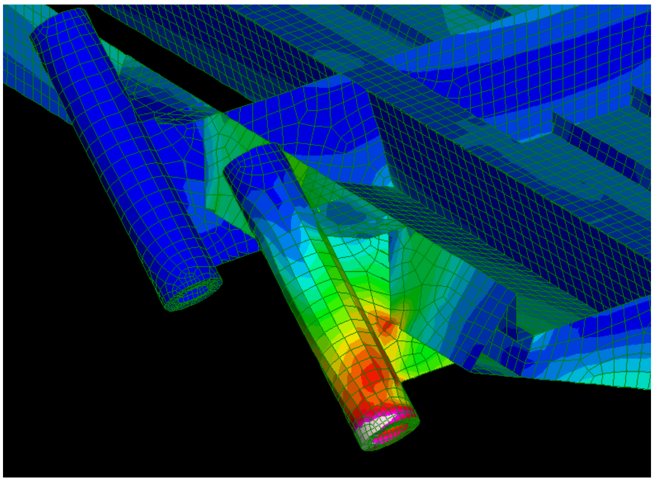


Detailed Design Digital Structural Model

Figure 12. Deck Stay Cable Anchorages

Stay cable anchorages are connected to the deck using fin plates welded to the girder edge beams. Steel outriggers at stay support connect the stays back to the deck box at internal diaphragm locations. The loads vary due to changing form pipe angles and cable loads. Anchorage forces are transmitted into the deck steelwork and composite concrete deck, with shear studs along the edge beam.

Backstays, comprising twin pairs of 43 strands, provide anchor loads from the main span to the Pier 3 tie-down support. These loads are transferred by heavy plate steelwork combined with reinforced concrete deck infill over the pier diaphragm.



FE Analysis of Back Stay Anchors

Detailed Design Digital Structural Model

Figure 13. Backstay Cable Anchorages

7. Construction Stage Analysis

Construction stage analysis refined stay cable sizes and tensioning loads to achieve final deck levels and optimize design. The method involved erecting 60m prefabricated modules on temporary supports, forming welded splices, casting the composite deck, assembling stay cables in sequence, and stressing in stages. Analysis using SOFiSTiK and MIDAS Civil software validated results, considering non-linear effects like second-order effects, cable sag, and time-dependent effects.

The analysis was complicated by the temporary portal supports used to erect the main span modules, which needed activation and deactivation at a specific “lift off” point. Some supports remained under load after stressing and required lowering with jacks to disengage. Key assumptions impacting the construction stage design, such as temporary works interface and loading assumptions, were recorded on the design drawings.

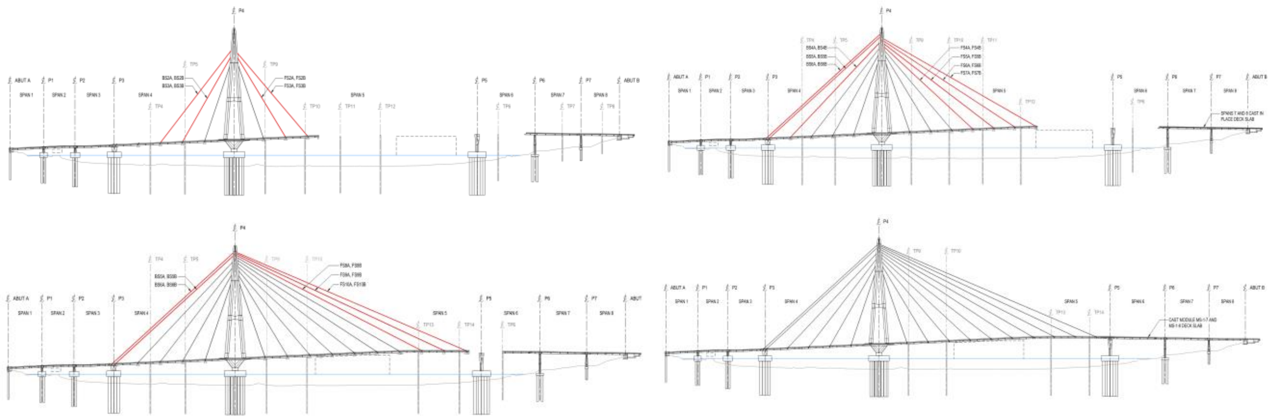


Figure 14. Typical Sequences for Main Span Construction

8. Cable Force Finding and Pre-camber

Cable force finding and deck level optimization were performed in SOFiSTiK using an iterative solver to determine the required stressing forces. This approach achieved force targets in the deck and mast while maintaining permanent compression in bridge bearings at Piers 5 and 6. A force target approach was used instead of a displacement target approach for better optimization. The design required moderate cambering, and the model was automatically updated with the pre-camber geometry to incorporate second-order effects.

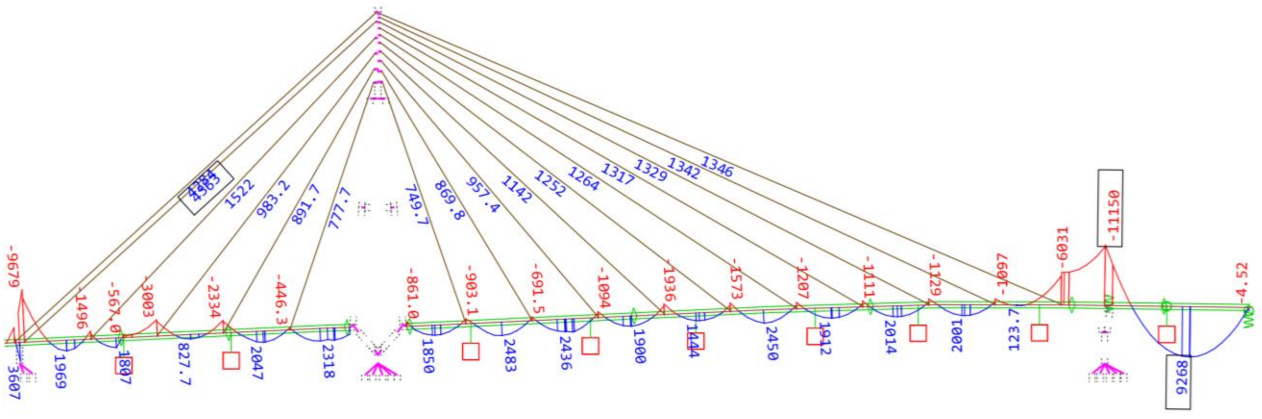


Figure 15. Optimised Permanent Effects Bending Moments and Cable Forces at Completion of Bridge.

Cable forces were checked against SLS and ULS limits of 50% and 74% of GUTS per Fib Bulletin 89. During construction, the SLS limit increased to 60% of GUTS per Eurocode

EN 1993-1-11. Once target forces were confirmed, deck pre-camber was determined for a “zero deflection” target at bridge completion. This accounts for tangential splicing of modules and vertical adjustment of temporary works portal levels.

Deflection control was refined during construction engineering, but stay stressing load, sequence, and deck module pre-camber were specified during design. The deck profile was surveyed during each cable stressing operation, with deflection goal criteria. The deflection control model was recalibrated during construction to reflect actual behaviour. Deck modules were weld spliced at varying levels, up to 500mm from the final vertical design position, following a target profile to achieve final levels.

9. Wind Studies

Long span lightweight pedestrian bridges are wind sensitive, so detailed wind studies were conducted by WSP and RWDI, including:

— Wind Study Specification: Defined wind study requirements and design inputs.

— Wind Climate Analysis: Documented site-specific wind speeds and turbulence.

— Desktop Aerodynamic Stability Assessment.

— Early stability indications using empirical calculations.

— Three Wind Tunnel tests, which assessed pylon aerodynamic characteristics, evaluated deck stability and aerodynamic coefficients, and provided a comprehensive aeroelastic model for 3D effects and stability, respectively.

— Numerical Wind Buffeting Analysis.

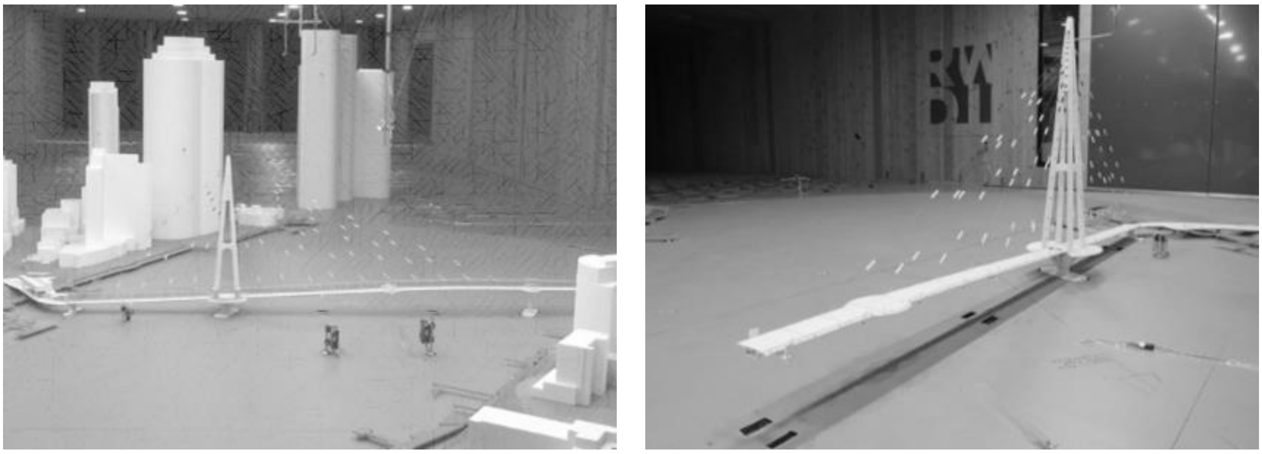
The numerical wind buffeting analysis aimed to correlate aeroelastic wind tunnel test results, determine displacements, accelerations, and force effects, and study wind effects in construction stages not covered in wind tunnel testing. Numerous design cases and construction stages were analysed, with a focus on the main span cantilevering to its maximum length before connecting to the Pier 5 module.

Wind parameters provided by the Wind Consultant included wind profile, turbulence properties, force coefficients, and flutter derivatives. Drag forces, vertical lift forces, and deck moments were derived from force coefficients. Separate force coefficients for each mast leg were used, accounting for the shielding effect of wind-facing legs. Two analysis methods were employed:

— Spectral Analysis (frequency domain): Efficient for predicting responses but limited for structural non-linearities and local effects.

— Transient Time History Analysis (time domain): Generated and analysed artificial wind histories, considering aerodynamic damping and galloping effects.

The analysis, based on unsteady theory and flutter derivatives, involved 10-minute runs with 12,000 steps each. 11-time history runs were conducted, and the mean of the maximum values was used as the maximum response.



Aeroelastic Wind Tunnel Model. Completed Bridge **Aeroelastic Wind Tunnel Model. Construction Stage**

Figure 16. Wind Tunnel Tests

Envelope plots of displacement, acceleration, and force histories were generated for both spectral and time history analysis to inform the bridge design.

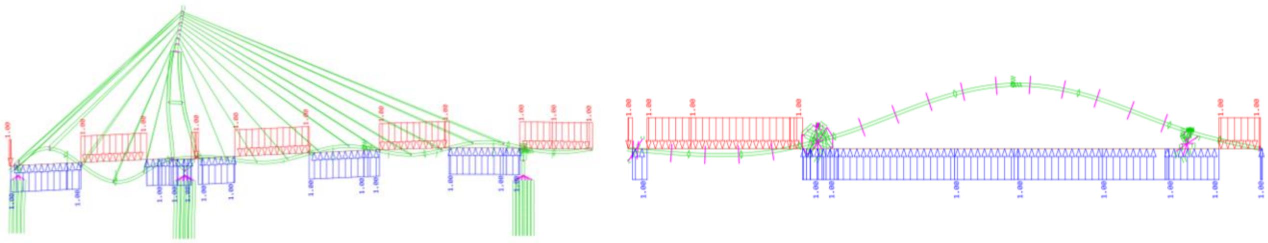
A close correlation was found between aeroelastic testing and transient buffeting analysis for deck vertical and mast longitudinal responses. The buffeting analysis also showed a higher transverse response of the mast, which was adopted for safe design. Deck accelerations under a mean wind of 15 m/s were evaluated for pedestrian comfort and found to be within limits, so no additional wind damping devices were needed.

10. Footfall Analysis

Dynamic assessment procedures and criteria were specified in the Performance Specification:

— Single person walking: CCIP-016 or SCI Publication P354 (vertical response),
— Crowd loading: SETRA Technical Guide (vertical and horizontal response),
— Crowd loading (lateral stability): London Millennium Footbridge, Dallard et al.

The single person load model resulted in a response factor well below the limit. SETRA defined frequency ranges for crowd loading resonance risk are 1.0 to 2.6Hz (vertical / longitudinal) and 0.3 to 1.3Hz (lateral). In this range, the maximum risk for resonance occurs between 1.7–2.1Hz and 0.5–1.1Hz, respectively. The load model applies a uniform load harmonically according to mode shapes. Two lateral modes and seven vertical/torsional modes required further consideration. Outside these higher risk ranges, a reduction factor was considered. Dynamic analysis confirmed accelerations were within comfort limits, so no additional damping was needed.



Key Vertical Mode (Elevation) **Key Horizontal Mode (Plan View)**

Figure 17. Example Harmonic Crowd Loading Model

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. The TMD was positioned within the box girder void in the main span with access restricted through a maintenance access hatch in the deck wearing surface.

Acknowledgments

This project was promoted by the Brisbane City Council. As Lead Consultant for bridge design, WSP worked with Connect Brisbane, a consortium of design, engineering and construction specialists led by BESIX Watpac, with Rizzani de Eccher, Dissing+ Weitling, Blight Rayner and ASPECT Studios. WSP's multidisciplinary approach included structural, geotechnical, mechanical, electrical, architectural, sustainability, landscaping, civil and temporary works.

The project took place from 2021 to 2024 and opened to the public in December 2024.

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