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Neville Bonner Bridge – Brisbane's Iconic New Pedestrian Link

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Abstract

The Neville Bonner Bridge is a unique hybrid arch-cable-stayed pedestrian bridge spanning 320m across the Brisbane River and constructed as part of the Queens Wharf Brisbane development.

The structure comprises a slender composite concrete deck supported with stay cables connected to the tapered steel arch and mast of the iconic white "ribbon" structure. With maximum clear span of 140m, and total height above the river of 75m, the structure was very slender and required considerable analytical analysis and design of all main components.

The objective of this paper is to describe the technical solution that evolved over the progressive design stages. Th design addressed the challenges of stage constructing the slender "ribbon" structure over the Brisbane River. The paper describes the flood resilience of the bridge foundation design including riverbed scour assessment. The Brisbane River is a major waterway, subject to maritime traffic, and a barge impact force was required to be designed for by the Port Authority.

Due to the slenderness of the deck structure, the bridge was identified to be susceptible to pedestrian footfall and wind dynamic effects, and the paper describes the dynamic analysis was undertaken to specify the requirements for tuned mass dampers.

All cables are designed with sufficient redundancy, and a specific time-history "accidental cable loss" analysis was undertaken. Cable dampers were required to four of the main deck cables to limit wind oscillations of the cables in service. Steel fabrication details are presented in the paper.

The design of a complex structure such as the Neville Bonner Bridge required a collaborative team effort. It cannot be overstated the importance of a collaborative approach not just within the design team, but also between the contractor, fabricators, designers, and stakeholders. The paper demonstrates that the Neville Bonner Bridge is an excellent example of what can be achieved when a team work together for a common outcome.

Keywords: Pedestrian Bridge, Arch-cable-Stayed, Queens Wharf, Ribbon Structure, Complex

1. Project background

Named after Australia's first Indigenous parliamentarian, the Neville Bonner Bridge is a new pedestrian bridge that crosses the Brisbane River and was commissioned by Destination Brisbane Consortium as part of the larger Queen's Wharf Integrated Resort Development (IRD).

The overall bridge structural form is a slender hybrid arch span and cable stayed main span; comprising a continuous tapered "ribbon" form of the combined arch and mast, which then anchored the primary deck cables. A pair of 48 strand backstay cables anchor the inclined mast segment to the top of the arch, anchoring the primary span through the southern leg of the arch to the Southbank abutment.

Access to the bridge at the southern abutment is provided by stairs and a pedestrian ramp at the South Bank Parklands, providing the main longitudinal anchor point of the bridge. The anchor tie back details defined the construction and overall form of the structure. At the northern side of the river, the bridge spans over the eight lane Riverside Expressway (REX) carriageway and is supported by the cantilevered podium structure of Level 4 of the Queen’s Wharf Integrated Resort Development.

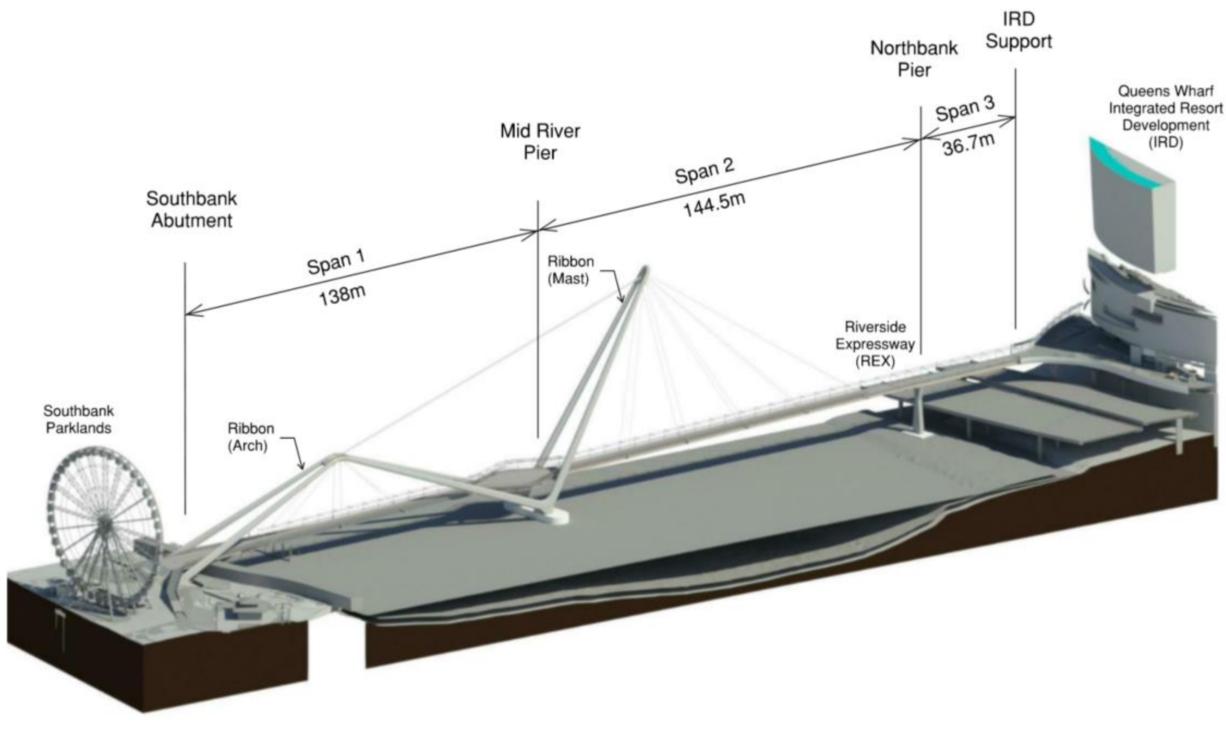
The deck is suspended from nine pairs of cables that connect to the ribbon at the top of the arch section and tip of the cantilevered mast. The “ribbon” is a fabricated steel structure, painted white and with night-time feature lighting – it not only provides practical benefits but also adds a striking visual element to Brisbane's skyline.

The structure is defined by three spans, with the southern arch-supported span being 138m, the main mast supported cable stayed span is 144.5m, with a shorter 36.7m end span over the REX which is in turn supported from the Level 4 structure of the Queens Wharf IRD. The bridge general arrangement is illustrated in Figure 1.

The vertical alignment has increasing grade from south to north, with 1V:21H grade (with landings) over Span 1; 1V:50H grade over the midspan landing area, and 1V:21H grade (with landings) over Span 2, then reducing to 1V:34H grade (no landings) over Span 3 – the bridge grading complies as a walkway to AS1428 Design for Access and Mobility. Horizontal alignment is straight from end to end, which simplifies the geometry and benefits the cable stayed support arrangement.

At the mid river pier, the deck widens in the central zone to provide additional area to encourage people to linger and enjoy views of the river, city and parklands. A continuous shade structure full length of the bridge provides sun protection.

Figure 1 Bridge arrangement



The bridge pedestrian pathway is 4.5m wide between balustrade railings and the deck is comprised of two welded steel I-girders each side, with a composite concrete deck slab, supported by discrete cable support beams typically spanning 22.8m between cable anchor points.

Vertical clearances were 4.5m minimum at the Southbank landing, 11.4m to the primary navigation channel in the Brisbane River and 6.7m above the Riverside Expressway.

2. Performance specification

The Neville Bonner Bridge required compliance in relation to Economic Development Queensland (EDQ) Development Conditions as part of the overall Priority Development Area of the Queens Wharf Redevelopment. The structural design was specified to comply with Australian Standard AS5100-2017 Bridge Design[1], and international standards as applicable.

International standards were adopted to address issues specific to long span cable stayed pedestrian bridges, as required to address issues relating to cable stay performance, wind performance, and pedestrian comfort from the light-weight bridge footfall dynamic response. Key international standards/guidelines referred to for the design included (for bridge dynamics): Eurocode 1 (EN1991-2:2003)[2] and supplemented by JRC Design of Lightweight Footbridges for Human Induced Vibrations (JRC 53442-2009)[3], and for stay cable system requirements: Fib Bulletin 89 Acceptance of Stay Cable Systems Using Prestressing Steels, 2019[4].

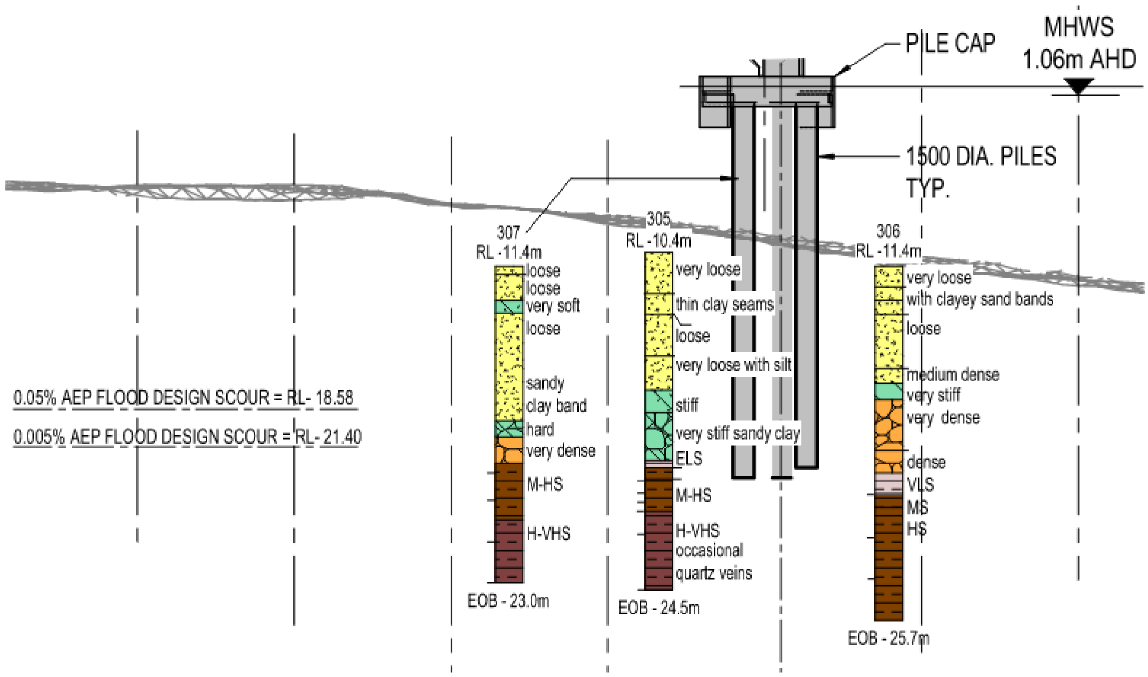
3. Foundation design

The site geological conditions are described as follows:

- The Southbank abutment consists of a variable cap of fill above the alluvium, underlain by variable weathered phyllite bedrock. At the Southern abutment, fill from existing surface is up to 3m deep, with alluvium approx. 20m deep to approx. RL-24m, underlain by weathered phyllite ("rock").
- At the mid river pier, the geotechnical profile generally consists of variable alluvium, underlain by variably weathered metasiltstone and phyllite ("rock"). The alluvium soils were encountered at the time investigation at depths of approximately 14 to 15m deep below riverbed level. The soils generally comprised interbedded layers of very loose to medium dense sands, very dense gravels and very soft to very stiff silty/sandy clays. The rock varied in strength between the bores from extremely low strength to very high strength.
- Northbank Pier foundations were constructed separately as part of the IRD foreshore works and comprised four 1200mm diameter bored piles that are integrated into the Queens Wharf Landing maritime structure.

For the geological conditions at the mid river pier, refer Figure 2.

Figure 2 Geological conditions at mid river pier



Piles at the Southbank Abutment consisted of 4 x 1200mm diameter driven steel tubes which included a concrete plug at 6m depth with reinforced concrete extending up to cut-off level. Secondary piles supporting the stair & walls enclosing the abutment included 5 x 600mm diameter driven steel tubes with reinforced concrete infill for the upper 4.5m above a concrete plug. For the driven steel tube piles, the effects of corrosion on the steel piles considered section reduction based on an annual steel corrosion rate multiplied by the design life of the bridge.

Piles at the Mid River Pier pile group consisted of 12 x 1500mm diameter cast in place piles with a temporary steel liner. These piles were specified with a 3m minimum socket length into rock founding material. The founding depth allowed for scour allowances in the Brisbane River under flood conditions.

The Mid River Pier pile group, along with the pile cap was modelled within the global finite element analysis model using beam elements and included spring support elements to represent the stiffness of the surrounding soil & rock. Initial spring stiffness values were based on preliminary geotechnical advice to generate spring reactions which were then provided to the geotechnical consultant for review, with final spring values reinserted back into the global structural model to achieve convergence of results with the geotechnical analysis model.

4. River scour assessment

A complex scour assessment in the zone around the piles was undertaken in accordance with the following standards & guidelines:

- Austroads (2018) Guide to Bridge Technology Part 8: Hydraulic Design of Waterway Structures[5]
- Departments of Transport and Main Roads (TMR) – Bridge Scour Manual – Supplement to Austroads Guide to Bridge Technology Part 8, Chapter 5: Bridge Scour (2018)[6]
- US Federal Highway Administration Hydraulics Engineering Circular No. 18[7].

Flood modelling was conducted by an external consultant, with the results of the 2D hydraulic model incorporated into the scour assessment. The total scour depths adopted for the design are comprised of the following three components:

1. Natural/static scour: Aggradation/degradation, natural channel migration or scour from confluences or bends
2. Contraction Scour: Erosion of the streambed at the bridge cross section due to the constriction/contraction of flow. Two design conditions, free flow and pressure flow conditions, were considered
3. Scour at Piers: Localised removal of material around the piers as a result of vortices induced by the obstructions to the flow.

Changes in upper and lower bound ground support conditions, including from scour allowances were included in the structural design as applicable to each limit state and load combination. Pile group & pile cap were orientated to reduce hydraulic drag forces due to river flow which also enabled the pile group to be arranged to provide increased resistance to the large lateral forces caused by vessel impact & lateral wind forces.

5. Vessel collision loads

Accidental collision from river vessels was considered in the design. The critical design case was a loaded 1500 tonne displacement barge under normal operating conditions (at maximum 10 knots operating velocity). The calculation of the barge collision load was established from the collision energy based on the approach prescribed in AASHTO LRFD (2007) Bridge Design Specifications (4th Edition, SI Units Version)[8] and equated to a 10 MN equivalent static impact force as an ultimate limit state collision load.

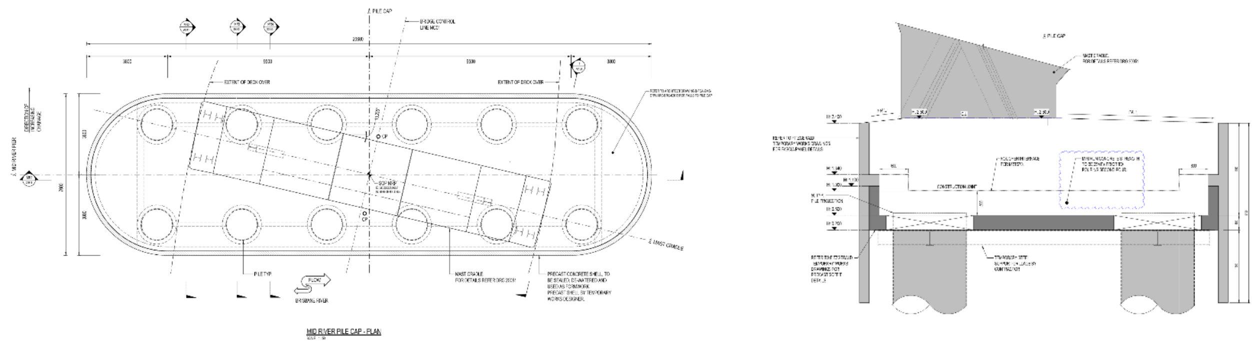
The design also considered the vessel collision case of maximum flood scour and river flood level, at the Mid River Pier - but in this case, the vessel velocity was reduced to the design flood velocity (representing the case of a loaded barge broken from its moorings in flood conditions). This case did not govern the design as the maximum flood velocity was significantly less than the maximum operating vessel velocity.

6. Pile caps

The Mid River Pier pile cap was constructed using a precast shell base and skirt permanent formwork arrangement. The shell was transported by barge and positioned over the piles on temporary steel supports with the base level of the shell set at mean sea level. The precast shell base was split into two segments for ease of lifting then sealed and dewatered. The pile cap was then poured in two stages to reduce design forces on the precast shells, with a 500mm deep initial pour and second 1500mm deep pour. Precast concrete skirts were cast into the second pour and detailed so that the toe remained below water level at low tide level, ensuring the piles are not exposed at low tide for visual and safety reasons.

The Mid River Pier pilecap arrangement is illustrated in Figure 3.

Figure 3 Mid River Pier Pilecap

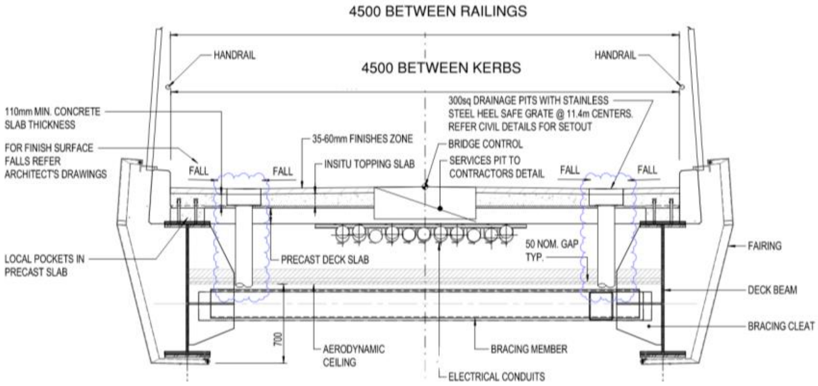


Plan view *Section view*

7. Deck design

The deck comprises a pair of steel I-shaped girders with precast deck units and a composite concrete topping slab. The concrete topping slab and steel girders are designed to act compositely with shear studs located within precast deck unit pockets aligned over the beam flanges and concrete filled as part of the topping slab construction. The typical deck section is illustrated in Figure 4.

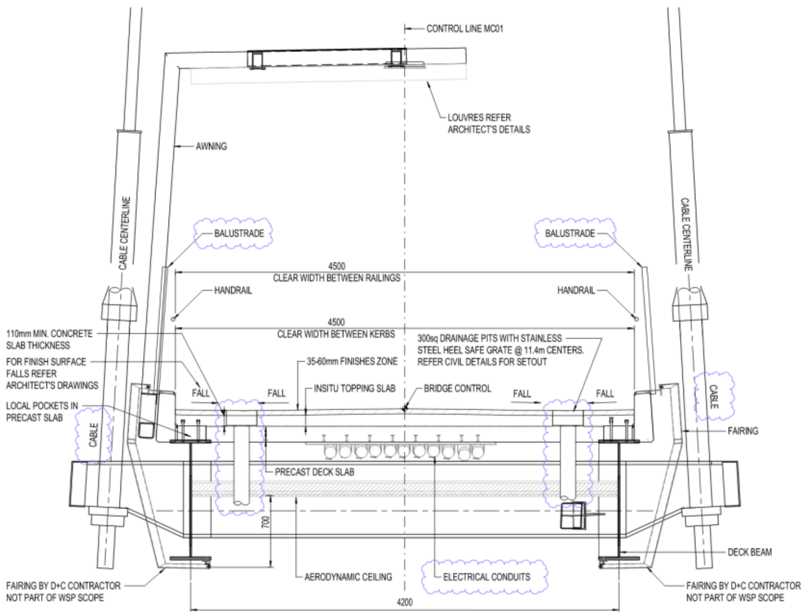
Figure 4 Typical deck section



The beams are tied together with continuous horizontal steel cross-bracing to provide lateral stability and stiffness. The deck structure is typically 1.5m deep overall inclusive of girder & slab, deck finishes and fairings, and provides a pedestrian path width of 4.5m clear between balustrade railings. The deck includes a shade canopy on one side for weather protection and has sufficient clearance for maintenance & emergency vehicles to access the structure. Electrical and communication utilities are located under the precast slab and are accessible through maintenance hatches from deck surface.

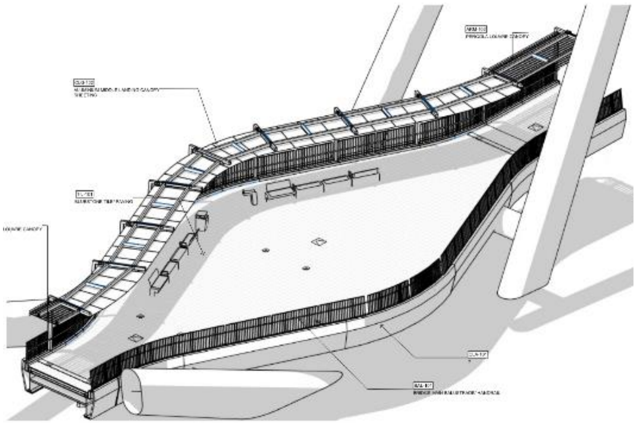
For the deck cross section showing cable support cross beams refer Figure 5.

Figure 5 Typical deck section at cable supports



At the halfway point along the bridge, there is a widened section of deck with seating which allows for scenic views up and down the river. The mid river deck widening is illustrated in Figure 6.

Figure 6 Mid river deck widening



Architectural Model (Revit)

Artist Impression (Source: Grimshaw Architects)

The deck elements were designed using the design actions taken from the global model and assessed against spreadsheet tools which verified sections and members capacities. The deck was assessed under construction stages which included cases for steel only (non-composite) prior to topping slab construction, and composite steel and concrete stages (after the topping slab had cured providing composite behaviour).

The girder design load effects also considered an accidental cable-loss load case – this considered load effects due to loss of a single primary supporting cable. The adopted design approach was based on Eurocode 3 – Design of Steel Structures Part 1-11 Design of Structures with Tension Components (2006)[9]. To further assess the appropriate dynamic response under accidental cable loss, and a rigorous time-history dynamic analysis was carried out at each cable location and these results further informed the design, with dynamic load amplification typically around 50% (Dynamic Factor = 1.5) being adopted at the critical cable support locations.

8. The Ribbon – Mast & Arch

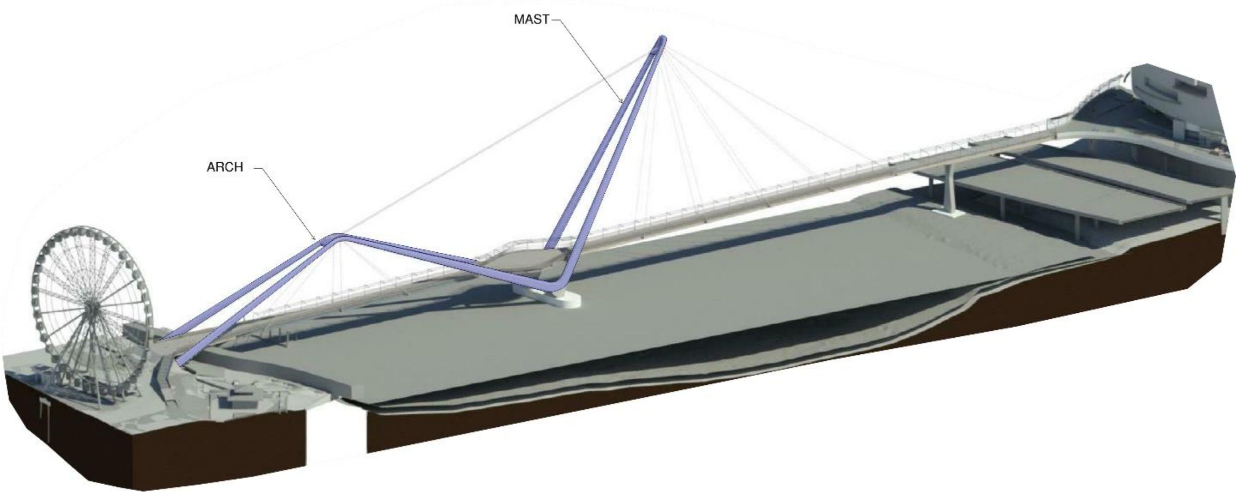
The "ribbon" refers to the mast and arch elements of the bridge and consist of a tapering 1.5m wide "pill" shape which tapers from 3m at the base to 1.5m deep at the top of the arch and mast. The arch provides support via "hanger" cables to the southern portion of the bridge deck between Southbank and the Mid River Pier whilst the mast provides support via stay cables to the northern portion of the bridge deck between the Mid River Pier and the Northbank Pier.

Following preliminary wind studies, the shape of the "ribbon" cross-section was optimised to reduce the effects of dynamic wind excitation due to synchronised vortex shedding. The shape was also designed to simplify fabrication by limiting the tapered components to the vertical sides of the section only. The curved base segments of the arch/mast base were fabricated from cut-down 1500mm x 20mm thick pipe segments which were induction bent to a very tight 7m radius to match the architectural concept of the bridge.

To provide increased resistance to lateral loading, the individual mast and arch legs are connected at the top of the with a rigid steel cross-section which 'portalised' the legs. These sections were analysed separately in local non-linear finite element analysis (FEA) models.

For an overall bridge 3D view highlighting the "ribbon" structure, refer Figure 7.

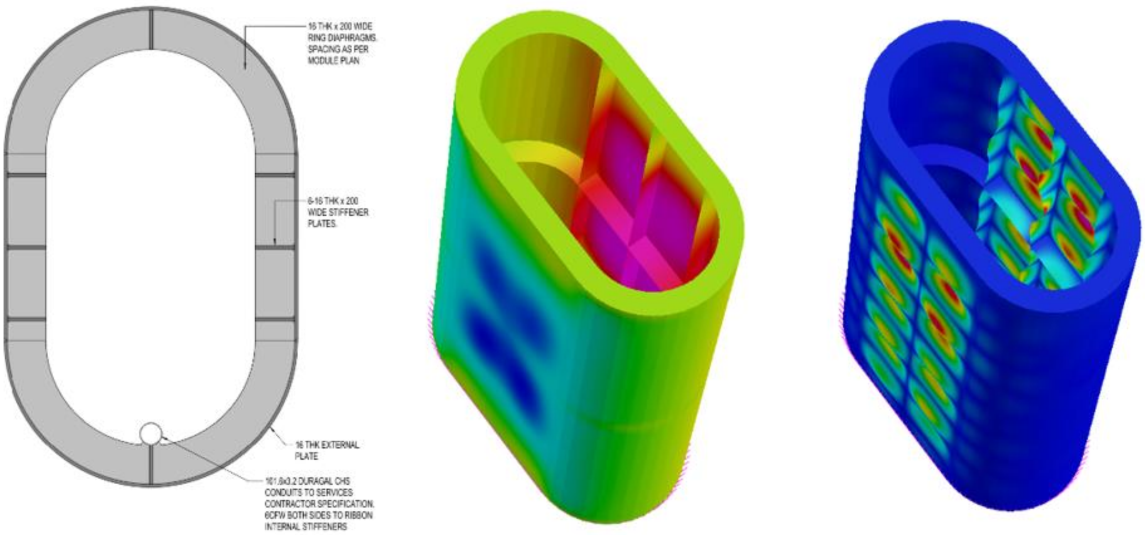
Figure 7 Overall bridge 3D view – "Ribbon" highlighted



The ribbon is formed from Grade 355 MPa steel specified to EN10025 rolled steel plate and stiffened with longitudinal stiffeners aligned along the longitudinal axis. Lateral restraint was provided to the longitudinal stiffeners by transverse ring diaphragm plates at 3m centres typically along the length of the ribbon to maintain the overall section stayed within slenderness limits. The ribbon platework arrangement was designed to AS5100.6[1] and verified using a series of local non-linear finite element models which assessed the platework buckling factors and stress levels against code limits.

The ribbon platework and internal stiffener arrangement underwent extensive optioneering with the aim of optimising steel tonnage and welding requirements. The 3D analysis complied with Section 4.10 of AS5100.6, which captured the effects of geometric imperfections and material non-linearity. The mast typical cross section with local FEA stress analysis and local FEA buckling analysis is illustrated in Figure 8.

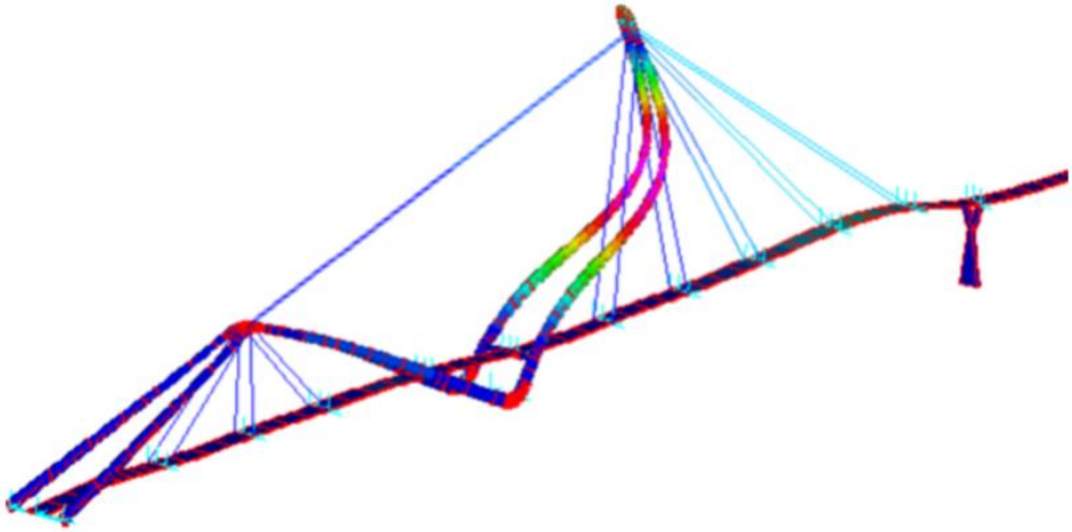
Figure 8 Mast typical cross-section, local FEA stress analysis, local FEA buckling analysis



Global buckling of the mast and arch elements was assessed via a non-linear elastic analysis using a global Strand7 non-linear buckling analysis model. The applied load cases included incremental loading up to several orders of magnitude above the ULS loads in order to observe the deformed shape at which the mast and arch would experience global buckling. Two key load cases were determined as being critical in the buckling analysis: ULS Permanent + Live Load and ULS Permanent + Wind Transverse Load. The two load cases produced significantly differing deflected shapes which in turn resulted in different effective lengths.

Refer Figure 9 image below which displays the buckling mode shapes.

Figure 9 Global buckling mode shapes for the ribbon

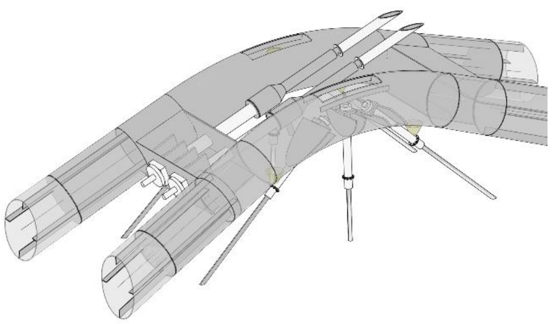


Imperfections were incorporated into the mast and arch member initial geometry in accordance with AS5100.6 Table 4.10.5 (A) & (B). The imperfection was applied in the direction to produce the worst effect.

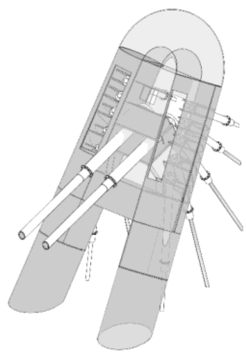
9. Cable node analysis

The "ribbon" cable anchorages are located at the top of the mast, and at the crown of the arch. These cable anchorage "nodes" feature complex geometry and significant platework to transfer loads from the anchorages to the ribbon outer platework. Cable anchorage 3D BIM model views are provided in Figure 10 to illustrate the structural arrangements.

Figure 10 Cable anchorages – Ribbon arch crown and mast head



Arch cable nodes

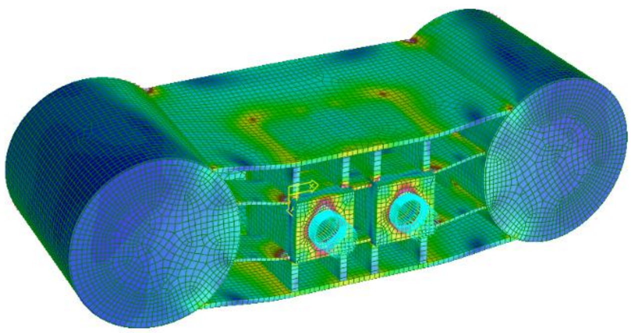


Mast cable nodes

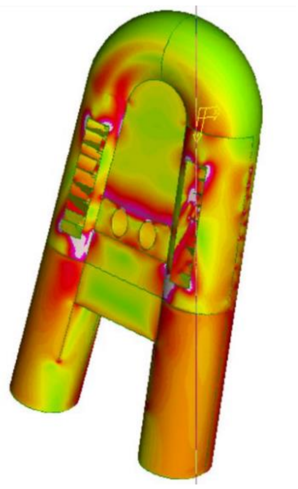
The local stresses were assessed using non-linear Strand7 finite element (plate element) models for each cable node to assess the peak stresses and plate buckling load factors. The remainder of the ribbon was modelled using line beam elements with rigid links at the plate interface in order to represent the frame actions transferred through the cable node. The analysis complied with Section 4.10 of AS5100.6[1] using linear material behaviour & geometric non-linearity. A mesh sensitivity analysis was carried out where mesh density was increased to the point where minimal change in plate deflections was apparent.

For the ribbon cable node finite element analysis, refer Figure 11.

Figure 11 Ribbon cable node FE analysis



Arch cable node & bearing plate Strand7



Mast cable node Strand7

The cable anchorages and immediate supporting platework was governed by 90% of the cable ultimate strength multiplied by an overstrength factor. This was to achieve ductility & robustness for accidental conditions (e.g. accidental cable loss case) in line with Fib Bulletin 89[4] and consistent with AASHTO LRFD[8].

10. Southbank abutment ribbon connection

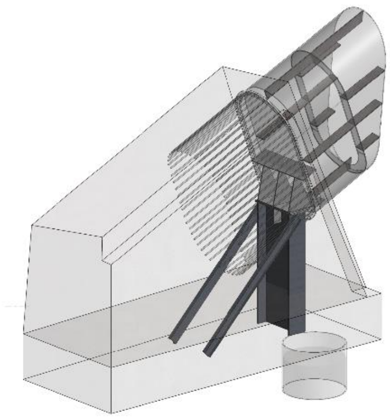
The Southbank Abutment includes the two arch thrust blocks which support each of the two arch legs, as well as a main central staircase for pedestrian access to the bridge from the Southbank precinct. The thrust blocks were sized to encompass the pile group, anchor the arch "ribbon" cross-section, and shaped to be angular and faceted for architectural purposes.

The thrust blocks were analysed using 3d 'brick' elements in Strand7 due to the complex shape, with the reinforcing requirements verified using a simple strut & tie analysis in accordance with AS5100.5[1].

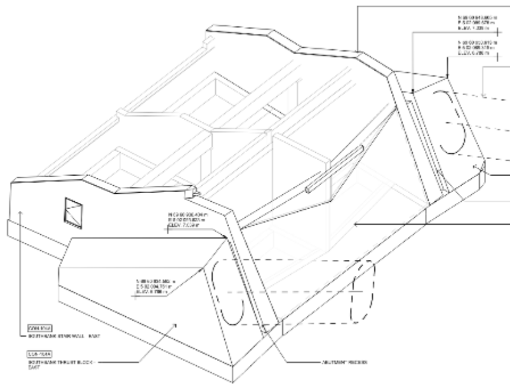
The arch legs were supported initially in the temporary case by a steel pedestal frame which was cast into the pile cap and allowed for vertical and horizontal adjustments prior to site welding. The arch ends included threaded couplers into which tail bars could be inserted which would anchor the arches into the concrete thrust blocks which were poured once all the arch legs were erected and aligned.

3D BIM model views for the Southbank abutment ribbon connection are illustrated in Figure 12.

Figure 12 Southbank Abutment ribbon connection



Southmank Abutment arch leg supported on temporary works pedestal & thrust block



Architectural isometric view of the Southbank Abutment

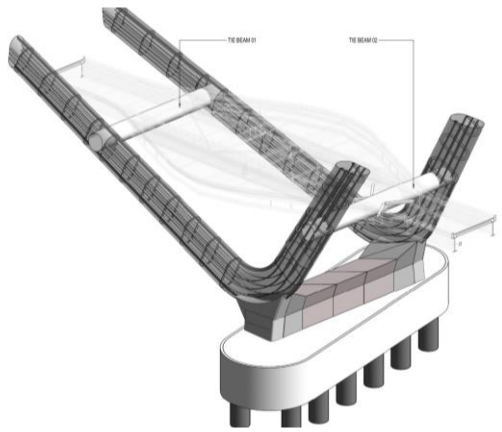
11. Mid River Pier ribbon connection

The "ribbon" is supported at the Mid River Pier pilecap on a reinforced concrete pedestal which sits directly on the pile cap. To reduce locked-in stresses into the pile cap from the erection staging of the ribbon and mast sections, the ribbon was supported on a temporary pinned connection. This was achieved using a steel support frame which was then embedded into the mast base pedestal concrete pour after initial stressing of the backstay cables.

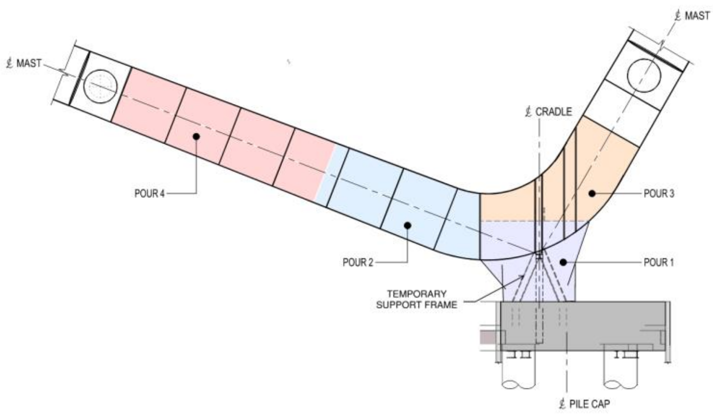
To improve impact resistance and prevent damage to the ribbon during flood events, the lower segments of the ribbon below deck level are concrete infilled and were poured in stages to reduce the wet concrete hydrostatic pressures on the ribbon steelwork.

The mid river pier ribbon connection showing a 3D view and the concrete pour staging sequence is illustrated in Figure 13.

Figure 13 Mid River Pier ribbon connection



3D view of ribbon pedestal



Temporary support and concrete pour sequence

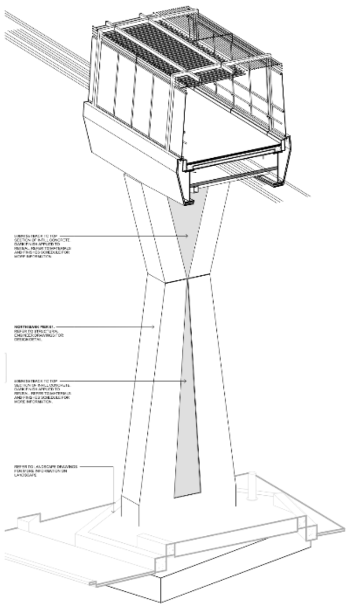
12. Northbank Pier support

The Northbank Pier is located on the Queen's Wharf side of the bridge within 'The Landing' western promontory precinct. The pier provides vertical and transverse support for the bridge deck and is released from the deck in the longitudinal direction via sliding-guided pot bearings.

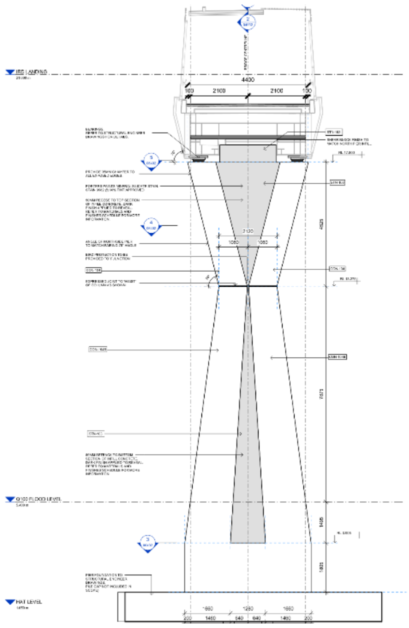
The deck to pier interface includes spatial provision for temporary jacks which will enable the bridge deck to be raised off its supports whilst the pot bearings are replaced at the end of their design life. Located at the top of the pier is a shear block which was designed to resist large accidental impact loads transferred from the deck to the pier. The shear key alleviated the shear demand on the pot bearing guides and hold-down bolts.

Northbank pier views from the 3D BIM model and the typical cross section are shown in Figure 14.

Figure 14 Northbank Pier



Isometric View of the Northbank Pier



Temporary Support and Concrete Pour Sequence

13. Stay cables

The stay cables are comprised of a total of 18 BBR Hi-Am Kona strand cables, made from 15.7mm diameter low relaxation individual strands with a Guaranteed Ultimate Tensile Strength (GUTS) of 1860MPa. Typical deck cables typically contain 12 strands each, and the twin main backstay cables contain 48 strands each. The first deck arch hanger cable set at the Southbank end contains 19 strands due to the increased forces caused by the larger deck length being supported. Stay cables were specified as a parallel strand system allowing for individual strand inspection, repair and replacement.

The cable anchorages were specified as proprietary type with the protection system for long term durability complying with Fib Bulletin 89[4].

Cable pipes were specified with double helical fillets for improved wind-rain vibration performance, with vandalism protection pipes at deck level. Fatigue testing requirements were also specified, in line with Fib Bulletin 89[4], with maximum SLS load target in the cable set at 50% UTS.

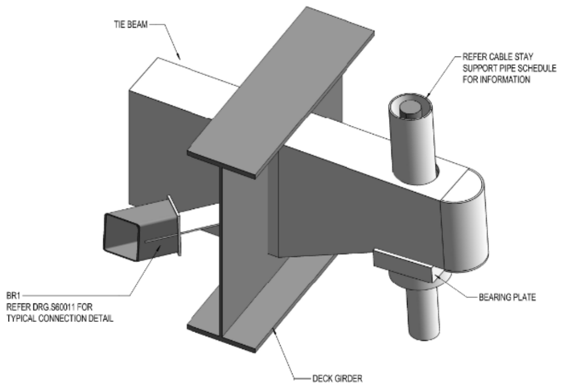
14. Deck anchorage design

At deck level, the stay cable anchorage load transfer is achieved using a fabricated RHS box beam which runs continuously through the deck girders, with girder web stiffeners provided to transfer the local box beam forces into the girder web. The box beam ends are tapered to minimise the overall projection of the cable anchorage assembly below deck level, for both aesthetic & accessibility reasons. The orientation of the beam box beam changed from cable to cable with the overall cross-sectional geometry kept consistent throughout.

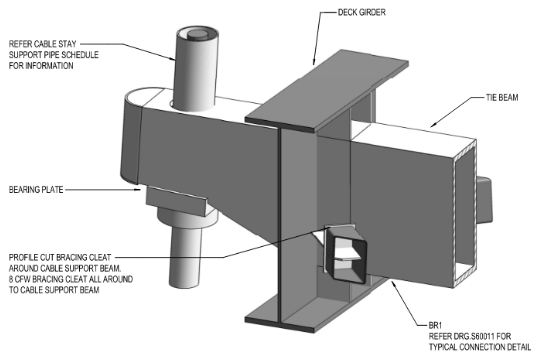
The deck level cable anchorages were also designed to resist Fib Bulletin 89[4] requirement of 90% of the cable ultimate tensile strength. This overload is applied to individual cables and is a ductility provision to ensure overstrength of the anchorage allowing post yield deformation of the cable.

3D representations of the deck cable anchorages are provided in Figure 15.

Figure 15 Cable anchorages – deck level



Cable to deck cross beam –Revit Model View 1



Cable to deck cross beam –Revit Model View 2

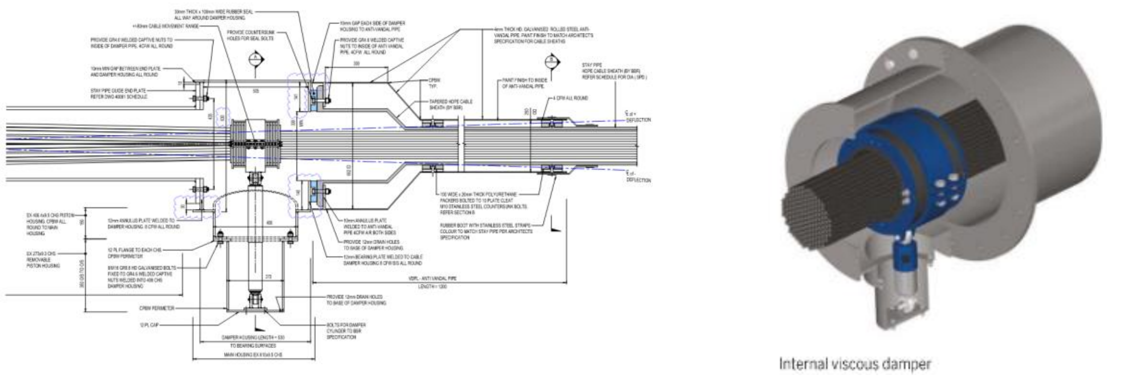
The cable anchorages typically consist of two parallel transfer plates that are designed to transfer the anchorage loads to the side walls of the mast and arch core steelwork. Each anchorage has a separate bearing plate and associated sealing plates to form a box section for load transfer. A steel stay guide pipe 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. The largest plate is 80mm thick for the main backstay cable anchorage, and 50 mm thick for the typical 12 and 19 strand cable anchorages.

15. Cable damping

The twin backstay cables and longer deck cables (cable sets 8 and 9) feature an internal viscous piston damper to reduce vibration caused by wind effects. The backstay cable dampers are located at the arch anchorage end, and cable set 8 and 9 dampers are located at the deck anchorage end. The dampers feature an internal damper configuration, as opposed to the external configuration, for aesthetic and durability purposes. The dampers are comprised of twin hydraulic telescopic cylinders where the inner cylinder has the piston working chamber and outer cylinder works as the housing and reservoir. The damper piston ends are connected to the cable deviator clamp and cable stay pipe.

Refer Figure 16 for images of the cable damper arrangement.

Figure 16 Cable dampers



Backstay cable internal viscous damper *Internal viscous damper (source: bbr)*

The main backstay cables also feature "cross-ties" between one-another located at approximate quarter points in order to reduce wind-induced wake galloping effects.

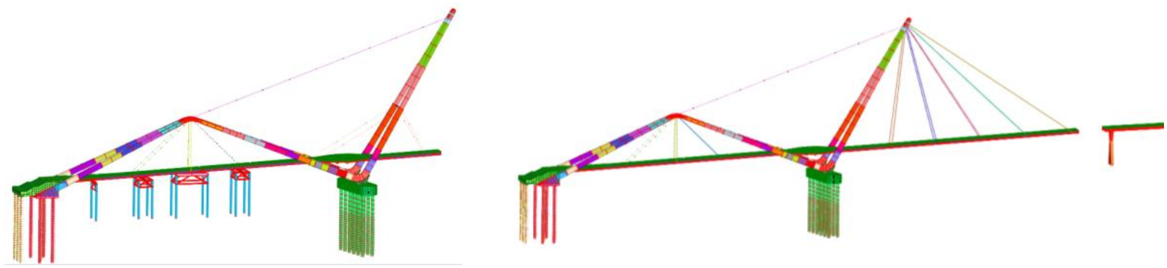
16. Construction stage analysis

A construction stage analysis was undertaken in order to quantify the stresses, strains and deformations which occur at different stages of construction. This analysis included the effects of the temporary propping and other falsework (such as temporary moment releases and ties) and was used to calculate the locked-in displacements and structural stresses due to the proposed construction staging.

Cable force finding and pre-strains were estimated using the Strand 7 staged analysis model, with deck levels set to be at design level under 1kPa live load and full dead load to ensure that deck gradients and deck deflected shape were acceptable under expected service conditions.

Example outputs from the Strand7 stage analysis are illustrated in Figure 17.

Figure 17 Stage analysis – Strand7 design model

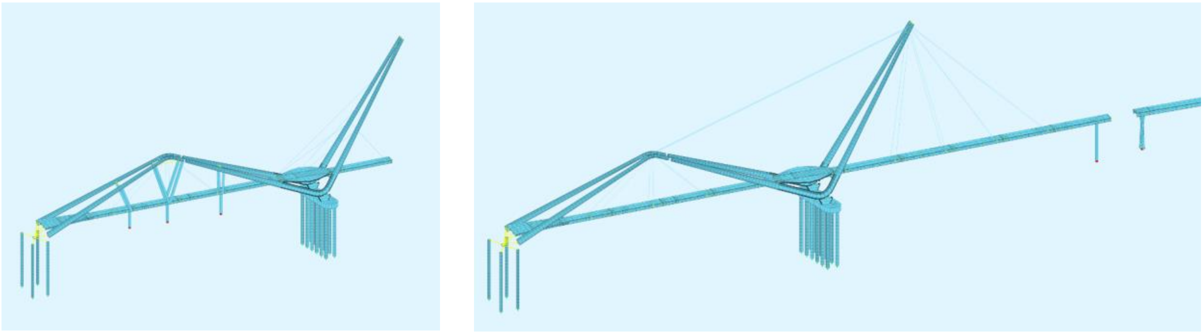


Construction stage 16 *Construction stage 23*

A key challenge within the stage analysis was determining the required level of cable and deck level preset to enable the final deck levels in the mast span to align with the design levels. One challenge was observed with the mast cable node which was observed to deflect significantly (up to 900mm) during installation of the main backstay cables. To minimise the impact of these deflections on the structure and to maintain acceptable alignment of the cable ends during erection, the two 48 strand backstay cables were stressed in three stages.

The design was verified using similar construction stage analysis in Sofistik software. In this case the cable force finding was performed as part of the construction stage analysis using the Unit-Force method. With this method the unknown cable forces are solved with a set of equations containing target forces or displacements. Non-linear effects such as second-order and third-order effects, cable sag and time dependent effects were solved through an internal iterative process. Illustrations of the stage analysis as independently verified using Sofistik software are provided in Figure 18.

Figure 18 Stage analysis and cable force finding – Sofistik verification model



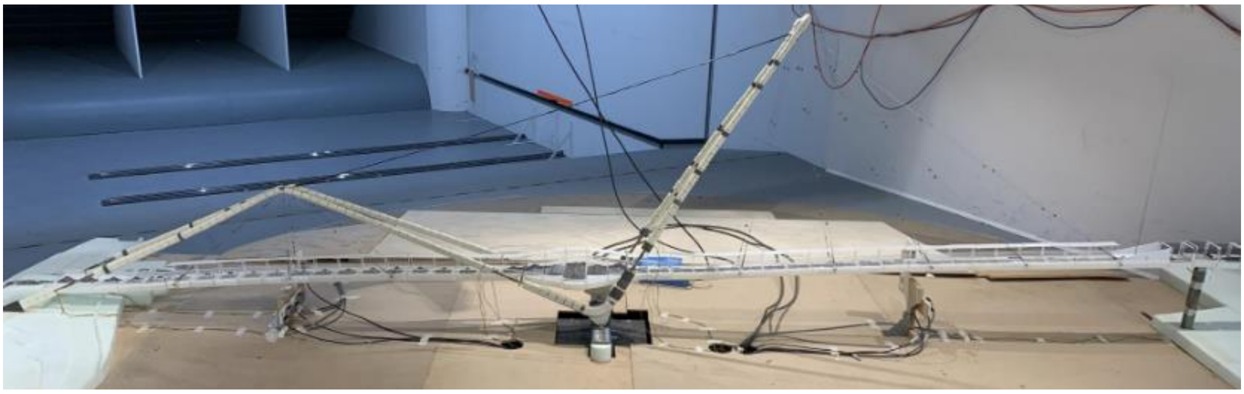
Construction stage 16 *Construction stage 23*

17. Wind studies

Due to the large spans, slenderness and lightweight characteristics of the bridge, detailed wind studies were undertaken to identify aspects of the bridge which are wind-sensitive and to validate the proposed design measures.

The wind studies included climate studies, section models of the deck and ribbon, and full aeroelastic models both in final condition and construction stages. The wind tunnel testing and reporting was carried by MEL Consulting.

Figure 19 Aeroelastic wind tunnel model



The deck sectional aerodynamic stability of the deck (for assessing vortex induced oscillations, galloping, flutter) provided design deck drag and lift coefficients. Some instabilities were initially found and was mitigated in the design arrangement, for example, the volume under the deck between girders was modified by positioning of a recessed ceiling, and outer fairings.

Full Aeroelastic Modelling of the bridge structure was undertaken using a 1/75 scale model. Aerodynamic stability of the bridge was measured in real time using laser measured displacements and strain gauges to capture estimated forces and moments.

During the wind analysis it was identified that supplementary damping was required in each of the arch & mast legs to achieve target damping levels and reducing structural motion due to the wind excitation. Each of the ribbon legs contained up to 4 individual hanging chains dampers which were housed inside a CHS tube. The chains hung centrally within the vertically aligned CHS tubes and were located within the arch and mast at the point of maximum modal displacement for the modes which required damping.

Additionally, tuned mass dampers (TMDs) specifically for wind effects were also added at two locations on the deck to provide an additional auxiliary damping of 2.5% under wind effects.

18. Footfall analysis

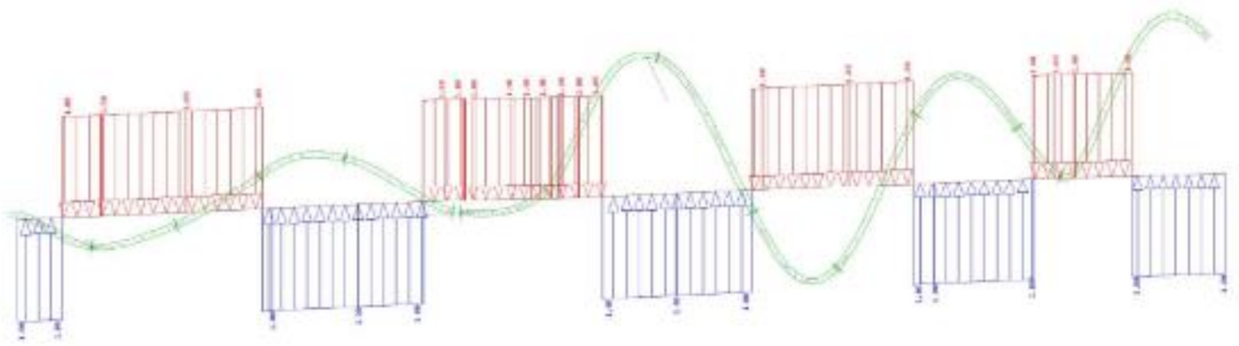
Footfall dynamic performance was assessed with reference to Eurocodes and the JRC Technical Report EUR 23984 EN - Design of Lightweight Footbridges for Human Induced Vibrations[3].

The footfall dynamics assessment methodology, adopted based on the JRC document, is summarized in the following steps:

- Determine the relevant pedestrian traffic classes measured in terms of pedestrian density
- Determine the relevant comfort classes measured in terms of acceleration criteria
- Apply the load associated to the relevant pedestrian traffic class harmonically
- Perform time domain dynamic analysis and determine the acceleration response
- Check response against the criteria (if criteria is not fulfilled, adjust the structure or apply damping measures)

The Sofistik 3D model was used to evaluate the dynamic performance of the bridge.

Figure 20 Excitation load applied according to mode shape



The results indicated that the vertical and horizontal accelerations (for selected comfort class to JRC) were exceeded and additional damping options were investigated. Supplementary damping was specified in order to reduce accelerations to acceptable levels. Three footfall Tuned Mass Dampers (TMDs) were implemented, two are combined vertical & horizontal dampers. The dampers were represented in the analysis model as structural masses connected to the deck via spring elements with damping properties assigned to them.

These dampers addressed footfall-induced accelerations and were additional to the wind damper requirements. The damping configuration adopted from the footfall dynamics assessment was also used to address deck-related wind dynamics issues which resulted from the aeroelastic wind tunnel assessment.

Bridge dynamic performance was verified by full-scale dynamic testing at the completion of construction with natural frequencies of the bridge measured using accelerometers which provided good correlation with the global FEA models. Following this, a series of single pedestrian and crowd walking tests took place across the bridge at various speeds. The resulting data recordings from the accelerometer indicated that the accelerations were well within the acceptable limits.

Figure 21 Full scale pedestrian footfall testing – single pedestrian and crowd walking

19. Construction

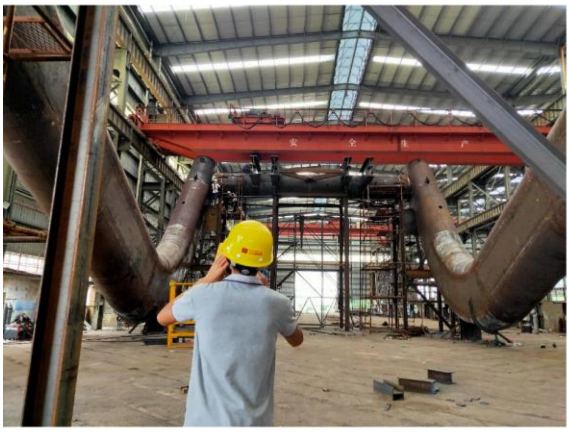
Construction of the bridge commenced in mid-2021 and was completed in late 2023. Fabrication of the main structural steel components was undertaken in Hong Kong by Goldwave and shipped by barge to the Pacific Tugs base at the Port of Brisbane. Bridge ribbon and deck modules were then welded together, pre-fitted with services and precast decking, painted and shipped by barge for erection on site.

Erection staging and temporary support locations were carefully considered and coordinated with the Contractor, Fitzgerald Constructions Australia, with module sizes for the ribbon sections sized to minimise falsework in the river. This resulted in some very large lifts with the main ribbon segments erected in eight individual components, with the largest component weighing up to 220t and 75m long in a single lift.

Final erection of the deck structure was completed in March 2023, with the final module landing within 20mm of design position longitudinally and precisely on design setout transverse to the span. The mast tip was within 7mm of design position, despite experiencing a movement range up to 900mm during earlier erection stages.

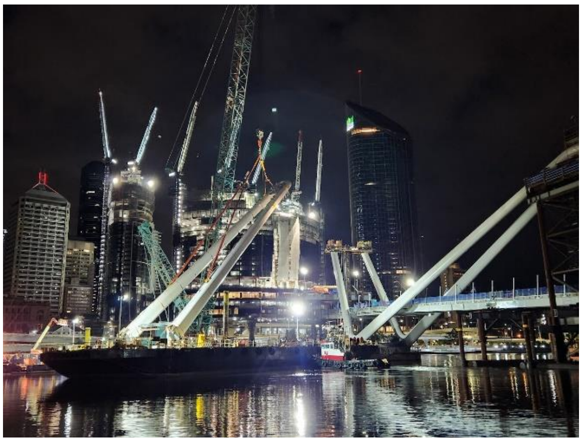
The precision and accuracy of construction and survey control meant that there was no need to do any final cable adjustments after the initial stressing. This is a remarkable feat of Contractor skill, given the size of the spans and range of deck movements that occurred during erection and also highlighted the accuracy of the FEA structural models used for design.

Figure 22 Bridge Construction Photos



Fabrication survey checks

Module 12 Deck Span above the REX



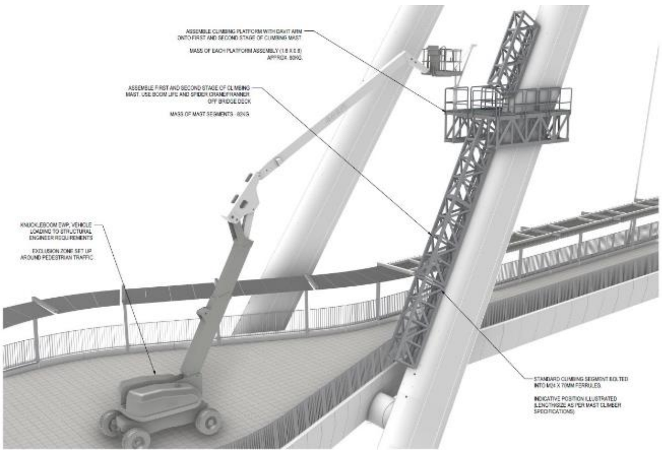
Mast Module Lift

Deck Construction

20. Maintenance access

Access to the high mast node is achieved via a 'mast-climber' which is a segmentally erected tower frame which is tied to the mast and includes a motorised working platform which moves up and down the mast. The climbing frame is fixed to the mast using temporary access ferrules.

Figure 23 3D Image of mast-climber



Access to the arch node is achieved via a knuckle boom or similar from the bridge deck level. Access to the cable anchorages at deck level will be via a temporary working platform attached to outside of the girders.

The top of the arch and mast include access panels which are screw fixed to the main ribbon sections and allow for temporary removal for maintenance inspections and access inside the top part of the ribbon in order to inspect the inside of the ribbon and the cable anchorages.

21. Conclusion

Neville Bonner Bridge is an iconic bridge providing a new cross river pedestrian link for the city of Brisbane. The technically challenging and novel design pushed the envelope of long-span cable stayed pedestrian bridges in Australia. With a slender deck profile and primary ribbon structure, the bridge demanded a high level of structural analysis, detailed considerations wind and dynamic response, and required precise control of deflections during construction.

The success of this project is a testimony to the successful collaboration between the Client (Destination Brisbane Consortium), the Contractor (Fitzgerald Constructions Australia) and consultant team.

22. References

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Acknowledgments

Named after the first Indigenous member of the Australian Parliament, Neville Bonner AO, the bridge is an integral part of the world class Queen's Wharf development. The bridge was designed and constructed for Destination Brisbane Consortium. The Design and Construct contract delivery team was led by Fitzgerald Constructions, with Grimshaw as lead architects, WSP as lead engineering design consultant, FSG as geotechnical engineers, MEL Consulting as wind consultants, and tuned mass damper testing and commissioning by Engineering Dynamics Pty Ltd.

(Source: https://www.wsp.com/en-au/news/2023/significant-step-forward-for-brisbane-pedestrian-bridge)

The Neville Bonner Bridge was opened to the public on 28 August 2024.