Conceptual design of the new bridge in Walyalup (Fremantle), the first precast segmental cable-stayed bridge in western Australia.

Diseño conceptual del nuevo puente de Walyalup (Fremantle), primer puente atirantado de dovela prefabricada en Australia

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ABSTRACT

The new bridge in Walyalup (Fremantle) replaces a 1939 timber bridge sharing the same footprint. Due to the very stringent project requirements and the different stakeholders demands, the project has struggled to achieve planning permission for a number of years. The solution finally approved required an innovative approach using a dual narrow segment built in balanced cantilever with a cable-stayed solution, allowing to minimize the closures of the existing bridge and meet the client and stakeholder requirements. The bridge typology provides the opportunity for high quality aesthetics and which resulted in the final shape and outer forms. The bridge is under construction with opening expected in late 2026.

RESUMEN

El nuevo puente en Walyalup (Fremantle) sobre el rio Swan reemplaza a un puente de madera de 1939 situado en la misma planta. Debido a la gran cantidad de restricciones de proyecto y las demandas de las diferentes autoridades, el proyecto había fracasado en lograr la aprobación ambiental en numerosas ocasiones en los últimos 20 años. La solución propuesta, una innovadora solución de dovelas estrechas construida paralela al puente existente mediante voladizo atirantado, permite minimizar los impactos en el puente existente y cumplir así los requisitos del cliente, con una alta calidad estética que condicionó las formas externas y dimensiones de los elementos estructurales principales.

KEYWORDS: precast segmental, cable-stayed, extrados, balanced cantilever.
PALABRAS CLAVE: dovela prefabricada, atirantado, extradorsal, construcción en voladizo.

1. Context. History, location and culture.

The mouth of the Bilya/river (also known as Swan river) meets the Indian ocean in Walyalup, which is a significant place of the Nyungar people, modernly known as Fremantle, west of Perth city. This area has been populated by Nyungar people for many thousands of years. After the initial arrival of Europeans in the seventeenth century, the first permanent crossing was built in 1866 (see Figure 1 below).

Figure 1. First Fremantle crossing, 1866

The current crossing at this location, built in 1939 with a projected lifespan of 40 years was a timber bridge like many others in this location (see figure 2 below).

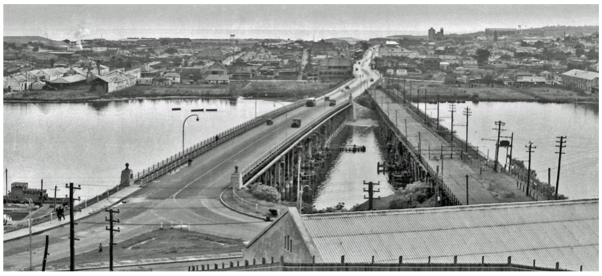

Figure 2. Current Fremantle crossing just after opening, 1940

The existing bridge has been subject to several retrofits and strengthening through the years, both at superstructure and substructure level and its capacity is currently compromised due to potential scour of the existing foundations, requiring constant inspection, along with providing an infrastructure unsuitable for modern active travel and the local demand which has led to several initiatives for its replacement for the last 20 years. A current view of the bridge can be seen in Figure 3 below.

Figure 3. Current Fremantle crossing in 2024

1.1 Culture and Place

During the different iterations to develop an alternative to the existing bridge, which have taken place over the last decade, it has been clear that the position of the bridge is heavily influenced by Cultural, Functional and Heritage parameters. The new crossing must address all these requirements simultaneously (see Figure 4 below).

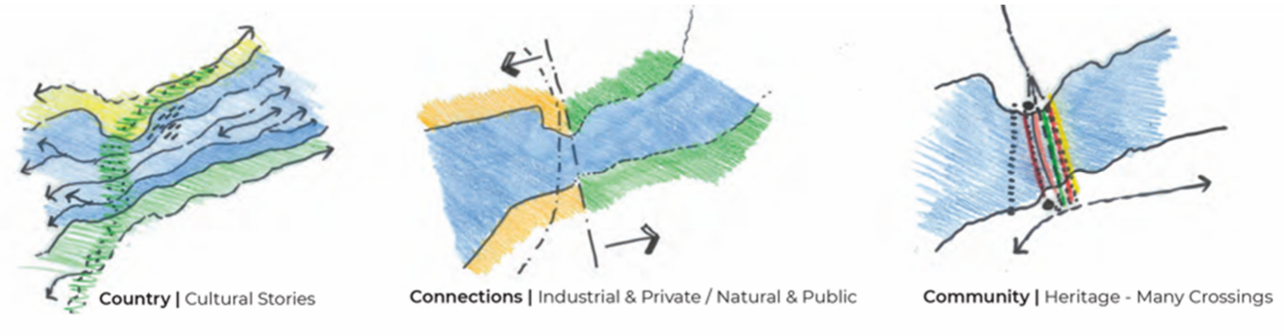

Figure 4. Site Cultural Context (Architecture, Urban & Landscape Design Framework).

The significance of the location of the existing bridge runs back for many generations of the Nyungar: in their mythology the Waakal (Rainbow Serpent) created the Bilya (now Swan River). In recent times the Fremantle Arts Bienale in 2021 had "Crossing" as a theme which resulted in the publication or Bilya Bidi-Crossings, an anthology of cultural and creative works responding to the specific site of the bridge [1], and Figure 5 below.

Figure 5. Authors of Bilya-Bidi/Crossings. FB 2021.

1.2 Engineering Constraints

From a structural point of view it is well known the existing bridge has limited scour protection, and its current behaviour is being monitored since the construction of the fenders for ship protection in 2015.

The vibrations induced by pile driving are limited to a threshold established by baseline measures and no works are permitted less than 4.5m from the existing structure on either side. This clearly limits the potential position of the foundations.

In addition, the final client, Main Roads Western Australia (MRWA), required a maximum closure of the existing bridge of 12 months (with 10 months or less desired).

Finally, the Southern link to Canning Highway, which is already substandard from a road design point of view, introduces a significant cost impact if the whole interchange needs to be upgraded (See Figure 6 below).

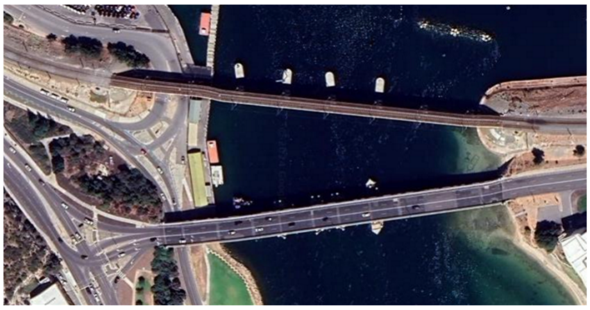

Figure 6. Aerial View (South to the left).

In terms of functional requirements, the new bridge must provide 2 lanes on each direction, plus a dual cycle lane in the west side and a shared path in the east side, which requires a minimum of 21m of deck width.

An additional challenge to be considered for any structural form and construction method is driven by the limitations of the local industry: Perth has a population of around 2.5 million (the whole Western Australia state of 2.8 million for an area of 2.5 million of square kilometres), and the next large city (over 200,000), Adelaide, is located 2,500 km away. This makes Perth one of the most isolated cities in the developed countries.

The local bridge construction industry has a limited availability of precast beams (U-shaped T-Roffs) and very little to no experience or availability of steel nor any kind of long span bridges.

1.2.1. Previous iterations

During the previous 10 years, different proposals have been developed without achieving planning successfully for different reasons, either cost, aesthetics or requiring an excessive closure time of the existing bridge.

These solutions varied from multi-span push-launched concrete deck built off line between the existing bridge and the rail bridge, which minimized the closure time but required a full redesign of Canning Highway junction and increased the cost beyond the Client's limits and did not achieve the aesthetic quality required by the local stake holders, to bowstring options or extrados located online over the existing footprint which achieved the aesthetic requirements but introduced durability and constructability issues (steel maintenance for the bowstring option) or very long closures for the extrados option.

In August 2023, after an intensive exercise of analysing numerous alternative solutions of different typologies, construction methods and locations, MRWA Finally selected as baseline a three span (69m-115m-69m) extrados solution with a concrete deck 2.5m deep, that in principle fulfilled the aesthetic requirements (see Figure 7), but had significant challenges from a construction point of view: the deck, located in the same footprint of the existing bridge, could not be built before demolishing the existing bridge, imposing a closure beyond the acceptable timeframe of 10-12 months.

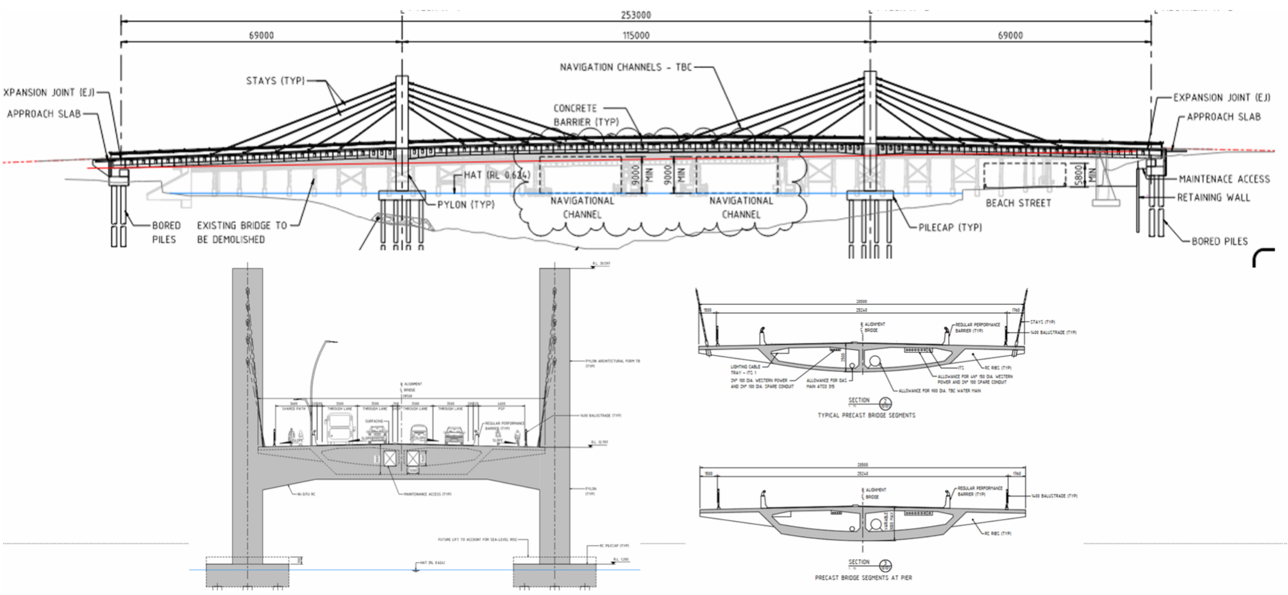

Figure 7. Baseline preferred option (2022- August 2023).

2. Design development.

The baseline design presented several challenges from a constructability point of view:

- The deck could not be erected prior to the demolition of the existing bridge, increasing the closure time of the existing road.
- The cable arrangement, to the edge of the box, required large cantilevers with ribs and a cornice beam to locate the anchors (see [2] for example), with difficult access to the cable anchors post installation for restressing.
- The cable plane inclination angle to the tower transversely plane generated bending on the tower requiring a significantly heavy cross section for this element.
- The box segments were required to be full width due to the location of the cables on both sides, making them extremely heavy if precast or complex to build if in situ methods were to be proposed.

In order to address the challenges indicated above, the following changes were introduced as part of the final proposal for planning.

- Replacement of the single three cell central box by two edge square boxes as narrow as possible, so the bridge width is not increased, without changing the transverse position of the stays, is governed by the functional requirements of the cross section and the distance from the foundation to the existing bridge.
- Increase of the box depth from 2.5m to 2.8m. This allows to operate inside the box for the cable stressing and restressing and the installation of the continuity prestress from a work environment isolated from the outer areas.
- Erection of the boxes in balance cantilever in advance of the closure of the existing bridge. Once the edge boxes are built, the transversal elements can be quickly erected using standard T-Roffs, precast U beams available locally).

This solution minimizes the closure time of the existing bridge, allowing the complex, unconventional with no local experience, precast segmental cable-stayed construction to be delivered prior to closing the existing bridge.

Figure 8. Edge box construction while keeping the existing bridge open.

Once the existing bridge is demolished, the remaining construction of the deck is extremely simple as the edge beams are at a constant distance of 22m with the installation of precast beams of equal spacing of approximately 6m and completed with a rectangular deck slab of constant depth of 0.30m.

2.1 Tower and deck aesthetics.

While the structural changes address the constructability and road closure requirements, they also bring some significant architectural challenges. A 2.8m deep box for a 120m main span gives a span depth ratio of L/42 which is not particularly slender for cable-stayed or even extrados bridges. Additionally, the external cantilever is minimal as the box is already providing a wider platform than the functional width required to avoid the cable to be very eccentric to the box.

Also, the tower-deck connection had to be integral to minimize maintenance, which makes the deck, tower and parapet form and relationship between them critical. The tower crosses the deck with the outer face slightly outside the outer web, giving priority to the tower over the box from an aesthetic point (see Figure 10 below).

Figure 9. Tower-Edge box-Parapet language (D+W Architects)

The box has a vertical web with a minimal 300mm cantilever and a 600mm deep parapet beam to minimize the visual impact of the vertical face of the box and the short cantilever (see Figure 11 below).

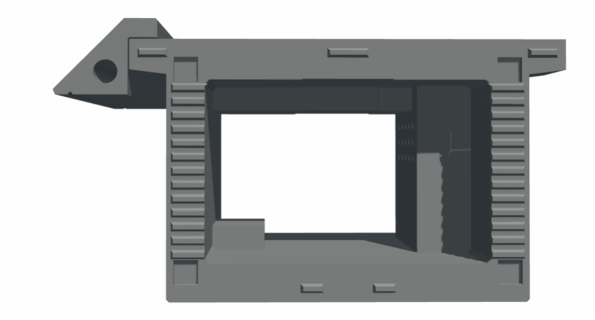

Figure 10. Segment and parapet render.

This solution was achieved after a a significant number of iterations on tower, deck and parapet options that were explored during the earlier stages of the design. Figure 12 below shows some of the early studies.

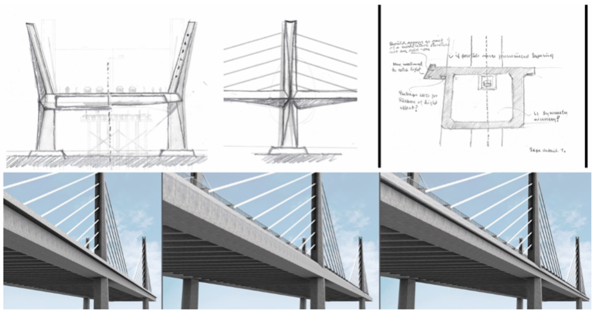

Figure 11. Hand sketches and renders of different tower and deck and parapet options

The solution finally chosen comprises a tower with a linear taper in the outside face from a rectangular shape at the top to a pentagonal section at foundation level. Due to the additional width provided by the box which allows for the cable system to be included in the tower width and inside the parapet railing, an additional area for pedestrians can be provided as shown in Figure 13. An architectural concrete collar has been designed around the cable at deck level.

2.2 Stay arrangement

Since the baseline was leaning towards an extrados solution, the final stay arrangement was considered as shallow as possible, at the angle between extrados and cable-stayed around 21-23 degrees. A harp arrangement was chosen to minimise the cable interference between the two planes of cables which are 23 m apart.

The cable spacing was chosen at 6m once the segment length was determined at 3m to the maximum width allowable for transport in Western Australia. This means that a stay is provided every second segment. The stay size varied between 53 strands for the longest tables to 40 for the shortest. As the main span is around 120m, a total number of 8 stays results in a first cable located at 12m in the fourth segment from the tower and the eighth stay in segment 18, leaving around 12m between stays at midspan.

This configuration provides a very balanced elevation which achieves the stakeholders desire of a high quality, iconic structure (see Figure 14 below).

Figure 12. Shared path showing additional width.

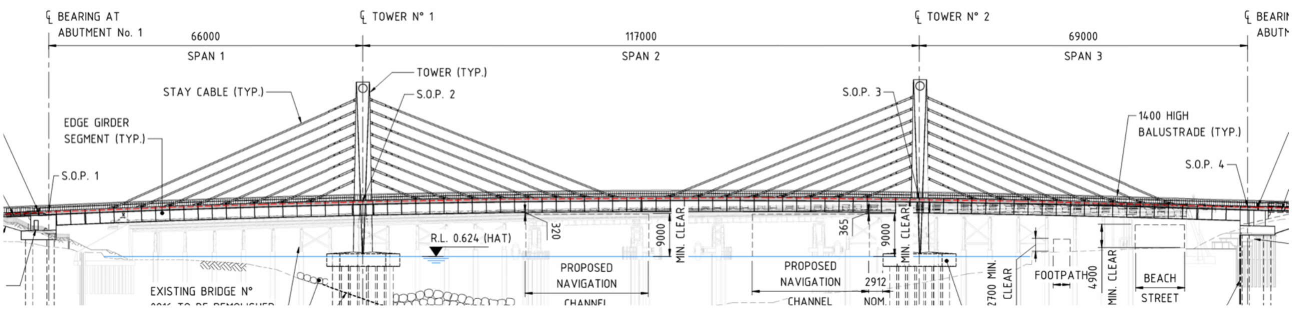

Figure 13. Bridge elevation. Final proposal

The transversal precast U beams (T-Troffs) are spaced 6m in alternate segments to the stays, this results in two typical segments, those supporting a stay and those connecting with a transversal beam. In this manner, as the segments are relatively small, an appropriate reinforcement can be provided to transfer the stay forces and the connection to the transversal elements which is provided via Macalloy Bars. The analysis for these elements is provided in a separate paper, see [3].

The deck is completed with a concrete slab 300mm deep, connected to the precast boxes via couplers, this connection plays a key role not only in the transversal behaviour but also in the longitudinal one as the slab acts as a very wide flange.

3. Structural behaviour

3.1 Deck. Longitudinal behaviour

The deck in the longitudinal direction acts as a box with the in-situ slab between edge beams acting as an eccentric flange (see Figure 15 below).

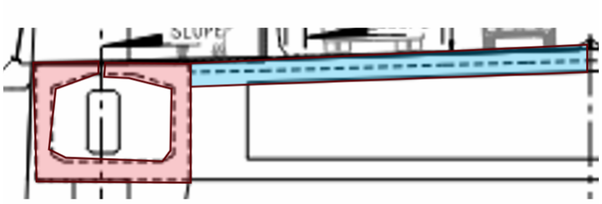

Figure 14. Half deck longitudinal section

From a modelling perspective, modern FEM packages allow to incorporate hybrid shell and beam elements eliminating the need for assumptions on effective width calculations. Later on, the stress limits (1Mpa minimum compression and 0.6 f_c maximum tension as per the Australian Code service condition [4]) can be checked directly from the frame elements used for the edge beam, while the Ultimate Capacity, if analysed using the frame for the edge elements only requires a significant amount of horizontal shear to be transferred at the connection. As an example, the following two plots show the traffic envelope unfactored for the edge beam (top) and the edge beam plus the slab integrated. The increase for the integral section is significant, particularly under positive (sag) bending (21.0MNm vs 31.8MNm)

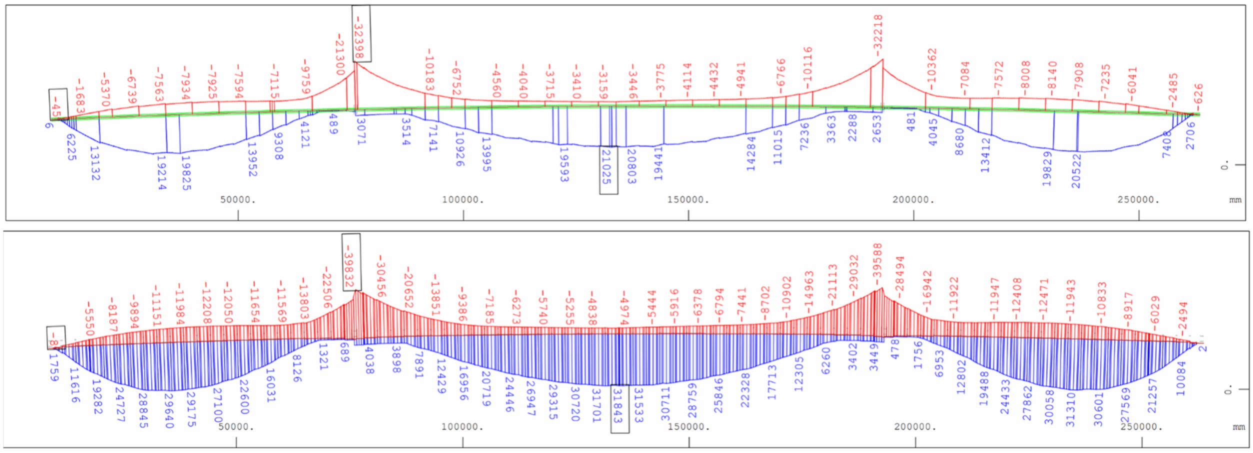

Figure 15. Bending moments due to traffic envelope. Edge beam only,(top) and edge beam + slab (bottom).

3.1.1 Stays and internal prestressing (PT)

As indicated in previous section the deck is not particularly slender with a span/depth ratio of L/42, which means that it will attract a significant bending under traffic load as the stay system will attract only part of the live load. Despite the cable angle of 22 degrees, the reality is that the deck behaves closer to an extrados behaviour with the stays taking stress ranges between 40 and 120MPa due to the full traffic load (the Australian traffic load unfactored is closer to the frequent load in the Eurocodes).

A conventional approach is to include internal continuity PT following the bending moment law to cater for the capacity required in Ultimate Limit Strength and the stresses to comply with the service requirements (see Figure 17 below).

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While this layout was developed to full compliance it presents significant disadvantages:

- Due to the stiffness of the deck and the cable system, the parasitic effects (hyperstatic moments) are very large, making the PT very inefficient.
- It requires to provide anchors for the continuity PT on many segments both with stays and ribs for the R-Roffs, complicating the internal formwork for the segments.
- As the segments are very small, moving heavy material inside the box, such as

multistrand jacks for the PT becomes a complex operation during construction.

A PT arrangement solution with straight cables running top and bottom, anchored at both sides of the tower and in the abutments, presents significant advantages, as all stressing points are located either at the abutment or besides the towers, where a temporary opening will be available during construction. The parasitic effects are minimised and the requirements for blisters in intermediate segments is removed.

This solution increases the total PT tonnage around 5%, but facilitates significantly the precast mould and the construction. A schematic layout for both options is shown in Figure 18 below. The temporary PT bars, required during the cantilever construction, are located in internal blisters top and bottom as most of the slab is already taken by the continuity PT ducts.

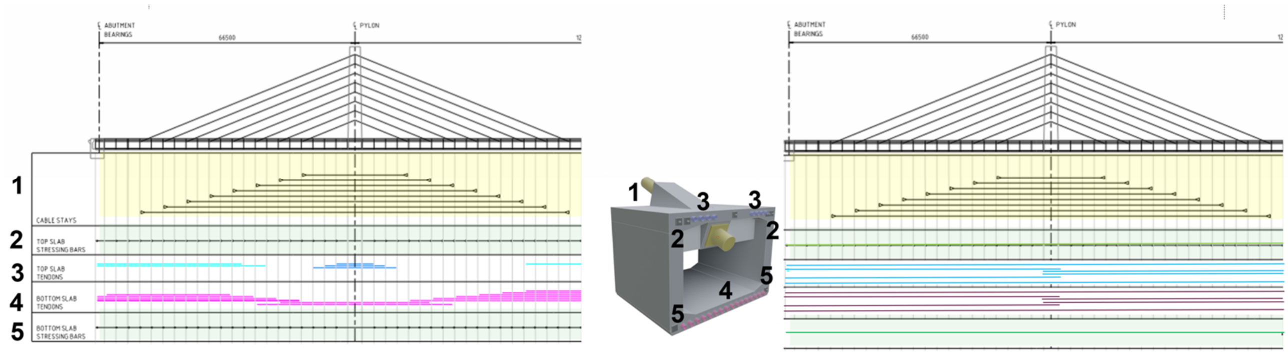

Figure 16. Conventional PT layout (left) and alternative PT layout (right).

3.1.2 Stay stressing

Another peculiarity of this bridge is the stay stressing. Generally, in cable-stayed construction, the cables are stressed only once and only specific cables might be stressed a second time. This can be achieved when the final cable force under permanent loads can be introduced during construction.

However, the construction method, as described in previous sections, only has weight of the edge box during the cantilever stages which represents around 40% of the permanent loads

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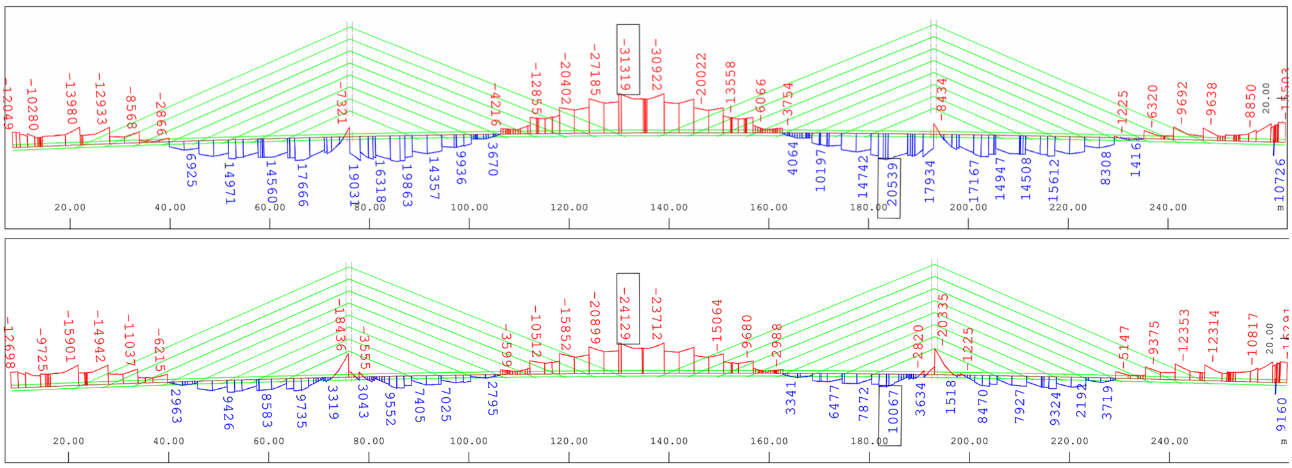

Figure 17. Bending moments of the edge girder at opening (top) and long term (bottom).

For this reason, a second stressing with a much larger force than the first stressing during the cantilever states is required in this bridge, Generally first stage stressing provides a final force in the Stays of around 1,800kN and the second stressing a final force between 5,000 and 5,800 depending on the cable. This final stressing, similar to the strategy followed in extrados bridges creates a bending moment law that counterbalances the live load, with hog at midspan and sage at supports. Despite the losses due to creep and shrinkage, this internal force distribution under permanent loads is necessary to optimize the stay and PT quantities while complying with the stress limits and ultimate limit state capacity of the deck. Figure 19 below shows the bending moment in the edge girder under permanent loads for the opening date and long term once creep and shrinkage have developed

3.1.3 Midspan Jacking

As indicated in previous sections, the connection between the tower and the deck was chosen as integral to minimize maintenance and avoid very large bearings under the towers with a clear visual impact. Due to the length of the span, 116 m and the construction method, this monolithic continuity introduces a significant bending in the towers and as a consequence, loads in the piles due to this portal frame effect.

This is due to the elastic shortening of the prestressing post closure at midspan, second cable stressing and creep and shrinkage. Although the piled foundations are relative long (up to 40m see Section 2.4.3 in [4]) with a equivalent fixed point around 15m deep from pilecap level or 25m from deck level, the portal frame effect is unavoidable. As in many arch structures (see [5] for example) this can be easily compensated by jacking the deck longitudinally at midspan before closing the central span. This jacking of around 450 tons, reduces the pile loads in around 1MN and compensates completely the elastic shortening and long-term effects.

3.2 Deck. Transversal behaviour

Transversally, the deck behaves as a precast beam 1.40m deep with a concrete slab flange 6m wide and 0.3m deep. The span to depth ratio of this element, with 1.7m vs. a 22m distance between inner faces of the edge boxes is L/12 suggesting that the precast beam is too deep. However the governing parameter is the connection to the edge beam, which is subject to both hog (due to the torsional stiffness of the box as expected) and sag (when the deck is eccentrically loaded, the difference in deflection between edge boxes creates sag in the edge beam to T-Roff connection that is closer to the live load. As the joint needs to work with a minimum compression of 1Mpa under service combinations, and the T-Roff does not allow for a large eccentricity on the Macalloy bars connecting the Edge box, this is the parameter that governs the depth of the T-Roff.

At the towers and in the adjacent segments, the T-Roff are 2.2m deep rather than 1.40m due to the portal frame effect in the transversal direction induced by the seismic load case.

3.3 Tower Foundations

From a conceptual point of view, the tower foundation was designed to provide a balance between aesthetic quality and structural efficiency. Due to the need for very long piles as the competent ground layers are very deep (see Ref to other paper), a piled foundation was cleary the correct solution. Also, the construction requirements forced for a minimum 0.5 m of exposed pilecap for high tide levels to prevent recreational boats from hitting the foundations. Although the tidal range in Perth is relatively low, this requirement would have created a significant visual impact on the foundation.

An asymmetric pile arrangement with 7 piles in a 4x3 configuration, following the pilecap shape and with a slight slope in the top face was proposed.

This solution does not only provides a high quality pilecap from a visual point of view but it also has advantages from a reinforcement detailing point of view as the bottom reinforcement does not need to pass through the piles vertical reinforcement in contiguous piles on perpendicular directions simultaneously.

4. Conclusions

The replacement of the existing timber bridge by a new bridge over the Swan river in Walyalup (Fremantle) which requires to satisfy simultaneously challenging requirements such as high aesthetic quality, cultural significance, traffic disruption on the existing bridge during construction and cost, has been extremely challenging, leading to rejected proposals for many years.

The solution developed by the Authors, with an innovative construction sequence consisting of two edge precast boxes providing a longitudinal structural system of a three span cable-stayed/extrados bridge that can be built before closing the traffic on the existing bridge, created a satisfactory solution that for which planning was granted in April 2024.

Figure 18. Precast pile-cap shell, February 2025.

The construction commenced in July 2024, with the first permanent pile driven in December 2024. The precast segments are under construction at the time of writing this paper and the erection of the edge girders is expected between June and November 2025 with the bridge opening in late 2026.

Aknowledgements

The authors would like to acknowledge the Nyungar people as the true custodians of the land of Walyalup. They pay respects to the Elders, both present and future and hope this project will be a positive contribution to Country.

The number of people involved in the project is significant and it would be very difficult to enumerate all contributors without risking leaving key people outside unfairly. This project would not be possible with a significant staff from ARUP and WSP apart from the Authors.

The final form and shape were achieved with a strong contribution from Anthony Duckworth providing invaluable local context along with Poul Ove Jensen, Jesper Henriksen and Danjiel Czorak from Dissing and Weitling as bridge architects.

The members of Alliance, on the construction side Laing O'Rourke and subcontracted VSL (cables and prestressing), CASE (erection engineering) and Brady's (marine works) for their constructive feedback to the constructability of the project.

Finally, to the technical staff from Main Roads Western Australia for their support and open-minded approach to this structural form which will be first kind in the State.

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