Análisis y comportamiento estructural del nuevo puente atirantado de dovelas prefabricadas sobre el río Swan en Walyalup (Fremantle, Australia).

Analysis and structural behavior of a precast segmental cable stayed bridge over the Swan River in Waylalup (Fremantle, Australia).

Marcos Sanchez Sanchez[a], **Andrea Castro Hernández**[b], **Lukasz Wojnarski**[c], **Ahmad Dimassi**[d], **Allan Cooke**[e], **Jose Manuel Martinez García**[f]

[a] Ingeniero de Caminos, Canales y Puertos. ARUP Ireland. Marcos.sanchez@arup.com [b] Ingeniera de Caminos, Canales y Puertos. WSP Spain. Andrea.castro@wsp.com [c] Bridge Engineer. ARUP UK. Lukasz.wojnarski@arup.com [d] Civil Engineer. WSP Australia. Ahmad.dimassi@wsp.com [e] Civil Engineer. Allan.cooke@arup.com [f] Doctor Ingeniero de Caminos, Canales y Puertos. WSP Spain. Josemanuel.MartinezGarcia@wsp.com

RESUMEN

El nuevo puente sobre el rio Swan en Walyalup (Fremantle, Perth), sustituye al antiguo puente de madera en la misma localización. La solución estructural propuesta consiste en dos cajones de hormigón prefabricado de dimensiones mínimas (2.80 de canto por 4.00m de ancho) separados 22 metros y unidos transversalmente por vigas prefabricadas cada 6 metros. El puente presenta luces de 66-117-69 metros con una configuración atirantada de plano vertical en arpa. Tanto la solución como el proceso constructivo están fuertemente condicionadas por los requisitos de proyecto de alta calidad estética y mantener el tráfico del puente existente. Este artículo describe los aspectos fundamentales del análisis estructural.

ABSTRACT

The new bridge over the Swan river in Fremantle, Perth, replaces an old timber bridge and it is located in the same footprint. The proposed structure, two precast boxes with minimal dimensions (2.80m deep and 4.00m wide) located in both edges of the deck, linked by a concrete slab and transversal beams every 6m. The bridge has a span arrangement of 66-117-69m with a stay system in harp arrangement. Both the structural solution and the construction process are heavily governed by the project constraints.

PALABRAS CLAVE: dovela prefabricada, atirantado, extradosado, voladizos sucesivos. Norma Australiana, cálculo sísmico.

KEYWORDS: Precast Segmental, cable stayed, extrados, balanced cantilever. Australian Standard. Seismic analysis.

1. General description of the structure

The new bridge over the Swan river estuary consists of a three-span cable-stayed bridge with a span distribution of 66-117-69m, and 8 stays per tower with a harp arrangement. It will be placed in the same footprint as the existing old timber bridge, which highly determines some of the structural solutions and construction procedures.

The overall height of the towers above the deck level is 25m, which results in a relatively shallow angle for the stays of around 22 degrees. This fact, combined with the relatively deep deck

Figure 1. Render of the Swan river bridge

(2.8m), designed to be able to stress and operate from inside a hollow box, results in a behaviour close to an extrados bridge.

The design, which was granted planning in April 2024 after numerous unsuccessful attempts for over a decade, was the result of a long process, which required a creative approach to achieve a high-quality solution while meeting the client requirements in terms of construction time and disruption on the existing structure. This process is described in a separate paper (see [1]).

Transversally, the deck accommodates 4 road traffic lanes and two shared-use paths on each side. It is 30m wide, with two edge precast box girders linked by a concrete slab and transverse beams every 6m. The boxes are 4m wide and 2.8m deep, with small overhangs to facilitate crane lifting and cantilever erection of the edge girders while allowing the traffic in the existing bridge to remain open.

Each cantilever edge girders are made of 19 segments 2.96m long with a stitch at mid-span 0.22m long. The abutment segments consist of 4 segments in the south and 3 segments in the north to the stitch.

The edge girders are transversally connected by precast U beams (T-roff in Western Australia) and are located every second segment, alternating with the stays segments, so that the anchor block of the stay and the T-roff is not located in the same segment. The height of the transverse beam is variable, to accommodate the transverse slope of the deck with a minimum height of 1.4m at the edge beam location.

At the tower locations and adjacent segments, the T-Roff beam is deeper, 2.2m to resist the concentration of transverse bending induced by the towers and foundations, particularly due to the seismic demand.

To complete the section, a 30cm deep concrete slab is cast on top of the transverse beams, which is also connected to the edge girders throughout the full length. See figure 2 below.

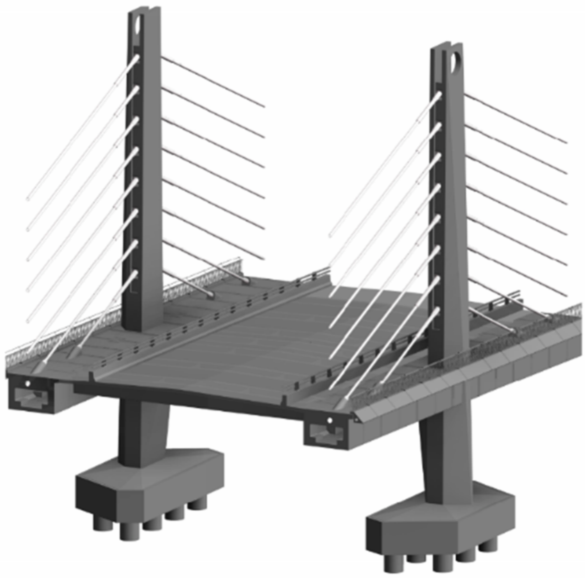

Figure 2. Isometric sectional view at tower 1

The towers are integral with the edge girders, with variable shape varying along their height. The foundations of the four towers consist of 7 piles 1.5m diameter in a 4+3 pile arrangement, which are partially above the river water level to avoid unintended collisions of the recreational boats in the area with the bridge.

The piles are composed of a steel casing, partially filled with reinforced concrete in the upper section and acting as a steel section in the lower section. They steel case will be driven in the riverbed to a maximum depth of 58m and have a free length that varies depending on the riverbed level and the ground conditions.

The foundation of the abutments consists of 4-pile foundation, for each edge girder, which are linked by an L-shaped wall, constructed at a later stage. This configuration allows the existing bridge to be opened to traffic while the foundations and edge girders are built, minimizing traffic disruption.

The edge girders are supported at the abutments on free sliding bearings. At each abutment, one of the edge girders has a longitudinal guide that is meant to fail under an earthquake event, during which a transverse shear key will engage instead.

2. Global analysis

2.1 General approach

The global analysis model has been developed using the software SOFiSTiK. The edge girders and transverse beams are modelled with beam elements, whereas the reinforced concrete slab is modelled by means of shell elements, which allows an accurate representation of the shear lag effects without requiring the calculation of effective widths that are used in grillages.

The slab is connected to the edge girders by means of a continuous rigid link, representing the actual interaction between the two elements and the horizontal shear transfer between the slab and the deck.

The stays are modelled as cable elements, attached to the edge girders at their actual anchor position, and to a node in the tower representing the top of each saddle. This assumption is valid, since it is a project requirement that the cable does not slip over the saddle at any moment, neither in the final configuration nor at any construction stage.

Towers and piers are modelled by means of beam elements, with varying cross sections. Considering the elongated shape of the tower pile caps in the longitudinal direction, they are modelled with shell elements, which better represent the semi-rigid behaviour of the foundation. The piles are modelled with beam elements as well, and the soil behaviour by means of linear springs in the horizontal and vertical directions.

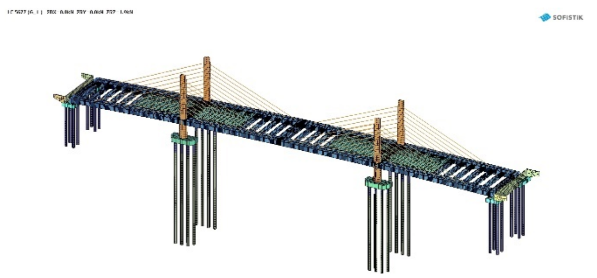

Figure 3. Global model. Construction stages during slab pouring.

The internal prestressing temporary stress bars that are required for the construction process are modelled with tendon features at their actual position and following a detailed construction stage sequence, developed in conjunction with the construction team.

Due to the nature of the project, where all the stakeholders are involved from the early stages, including the architect, construction teams and final client, a vast number of alternatives have been analysed in terms of geometry and construction procedures.

Due to the iterative nature of the design development a fully parametric automated workflow connecting Excel and Sofistik automatically was specifically developed.

One of the key challenges of the design is the cable stay tuning to optimize the cable forces and internal prestressing layout and quantities. The two structural systems: the edge beam working as a portal frame in longitudinal bending, stiffened by the top slab and the stay system are strongly coupled and are of similar effectiveness.

This requires a fine balance between stay forces and internal prestressing which is not obvious. The structural system was refined to minimize the main member sizes, avoiding an unnecessarily wide bridge or a box that impacts the traffic on the existing bridge.

All the traffic and transient loads have been determined applying Australian Standard AS5100.2:2017 [2] and the heavy vehicles requested by Mainroads Western Australia, and also a seismic and ship impact load cases.

The seismic analysis performed, as indicated in the Australian Standard is based on modal superposition with cracked section properties as indicated in Clause 15.2.1 of AS5100.2:2017 [2]

Since the structure is integral, the soil-structure integration requires specific calibration. The global model has been analysed using upper and lower bound ground stiffness, including pile group effects. The soil-structure interaction is not only relevant for the substructure design but also for the superstructure, since the deck is integral with the towers, and the portal frame effects are significant.

2.2 Superstructure. Longitudinal beams

2.1.1. Shear lag and distribution of forces between slab and edge box.

Due to the configuration of the bridge, the main resisting section for longitudinal forces is a composite section that includes the two post-tensioned edge girders and a 22m wide, 0.3m thick reinforced concrete slab.

The distribution of longitudinal forces is highly dependent on the stressing sequence of the stays and tendons. A tendon or stay that is stressed before the slab is cast provides axial compression and bending moment in the edge girders, whereas the force will be distributed between the slab and the girder if the slab is already casted.

Moreover, the stressing of one of the stays closer to mid-span with the full slab casted will create axial tension in the slab which needs to be carefully considered.

As indicated in the conceptual design paper (see [1]) the main stays can be only partially stressed during the cantilever stage as the part of the self-weight present during that stage is less than 40% of the permanent loads, which lead to a two-stage stressing for every cable (See reference [1], section 3.1.2).

2.1.2. Calibration of the Creep and Shrinkage curves.

The structure comprises a combination of precast and cast-in-situ concrete elements. The age of the different elements when they are installed as well as the age of loading is fundamental to estimating the creep and shrinkage effects on the structure as stress redistribution will take place both at sectional level and globally.

Creep and shrinkage curves have been calibrated using the provisions provided in the Australian standard AS5100.5:2017 [3], which required separated simplified models due to the fact that the Australian standards produce graphic curves where the humidity and equivalent thickness are embedded in the formulation.

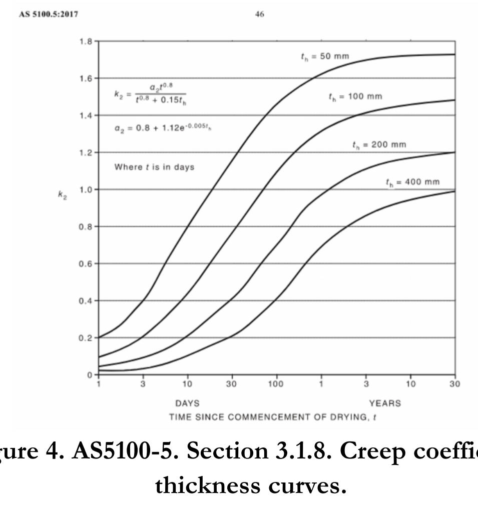

Figure 4. AS5100-5. Section 3.1.8. Creep coefficient, thickness curves.

A consequence of the combination of structural elements is the transfer of compression forces that occurs in the long term due to creep, between the post-tensioned edge girders and the concrete slab. On the bridge opening day, the edge girders are heavily compressed, while the slab is in tension, even at midspan, due to the stressing of the longer cables. In the long term, part of the compression in the edge girders is lost and transferred to the slab. It can be observed in the figures below that the slab is almost fully in compression. This axial effect is coupled with the bending redistribution. The figures below show the axial distribution in the edge beam and slab at opening and at long term.

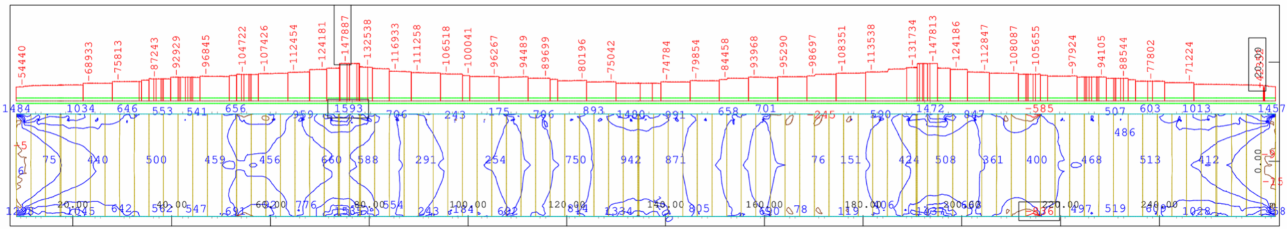

Figure 5. Axial load in the edge girder (top red, in KN) and in the slab (KN/m) at opening.

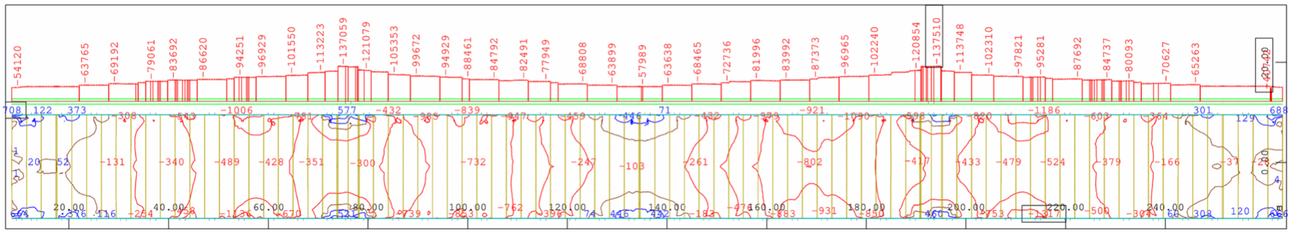

Figure 6. Axial load in the edge girder (top red, in KN) and in the slab (KN/m) at long term.

As shown in the figures above, at opening the axial in the edge box varies between 147MN close to supports and 74MN at midspan on the main span. This force is reduced to 137MN at supports and 58MN at midspan after creep and shrinkage has occurred. However, the vast majority of this effect is not due to the portal frame effect but due to the transfer between the edge box and the slab: at opening, the deck slab is in tension, with values varying between 940kN/m at midspan due to a hog bending to 240kN/m at quarter of the span and 600kN/m over the towers.

This effect is highly sensitive to the segment ages and the real stiffness of the concrete slab.

2.1.3 Internal Post-tension (PT)

The post-tension system is greatly constrained by the reduced dimensions of the interior of the edge box girders. Stays are located every other segment, with transverse beams installed in alternate segments. Therefore, each segment has either an anchor block for the stay or for the transverse beam, making it complicated to locate the blisters for the tendons.

With the limitations mentioned above, and as indicated in the conceptual design paper [1] a prestress layout with anchors located only at the towers and the abutments has been designed.

2.1.4 Governing Criteria SLS and ULS Traffic.

For the Service limit state, the design criteria request a minimum compression of 1Mpa at the segmental joints as per AS5100.5:2017 [3] clause 8.6.2.2. and a maximum compression of 0.6 fc. Since the concrete used for the segment is C65, this results in a maximum compression of 39Mpa.

During the cantilever construction, the criteria agreed with the client was similar to the one included in AASTHO [4], requiring decompression for the construction loads without environmental effects (thermal and wind) and allowing a transient tension stress of 2Mpa during when the envelope of thermal and wind for a return period of 20 years is considered.

For the reinforced concrete slab, cracking is controlled by limiting the stress in the reinforcement as per clause 8.6.1 of AS5100.5:2017 [3].

Due to the configuration of the deck and the construction process, it is critical to control the stresses on each of the four corners of the edge box as the transversal bending effects are not insignificant (see figure 7 for example, where more than 2Mpa of difference between corners at the same level can be observed).

As expected, the critical stressed are achieved at opening date for the maximum compression and in the long term for the minimum compression. The following figure shows the stresses in the bottom flange for these two scenarios.

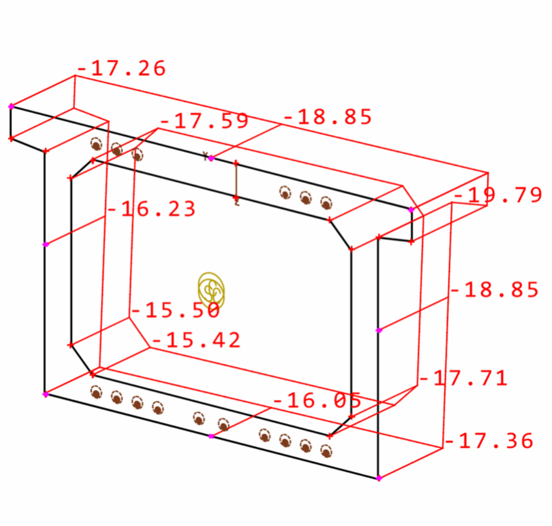

Figure 7. Stresses of segment 13 at opening

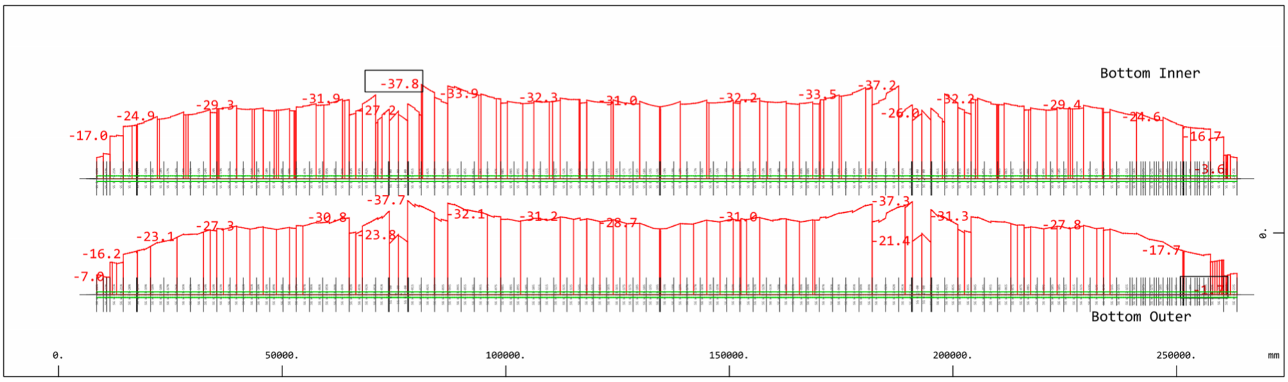

Figure 8. Longitudinal stresses bottom flange My min envelope at opening

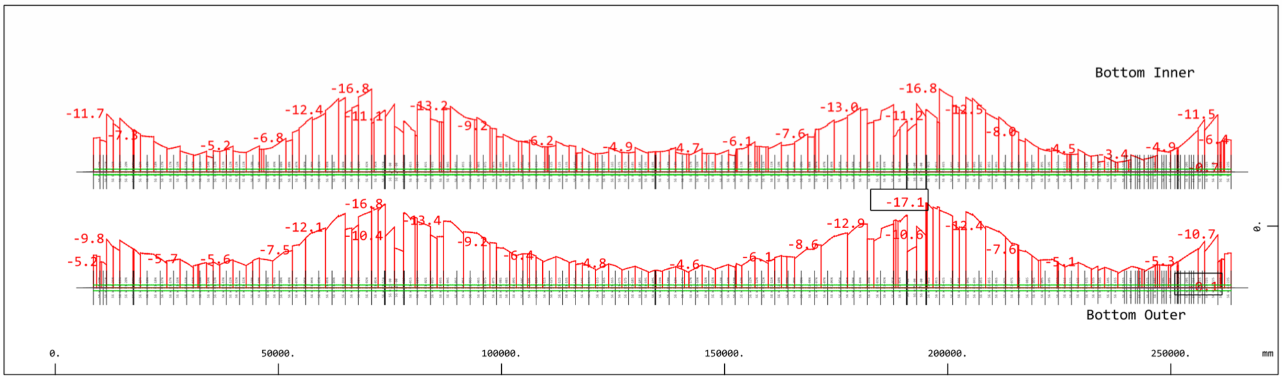

Figure 9. Longitudinal stresses bottom flange My max envelope at long term.

In ultimate limit state, the edge girders are designed for bending disregarding the contribution of the longitudinal passive reinforcement, since it does not cross the segmental joints. Surprisingly, due to the configuration of the bridge which combines a relatively stiff portal frame effect with a cable stayed action which has been designed similar to an extrados bridge, with a reverse of the bending moment law under permanent loads (see Section 3 of [1]) the capacity under Ultimate limit state is also close to the maximum demand. It is also remarkable that the high level of compression introduced by the internal PT and the cable system makes most of the sections achieve their maximum capacity under the compression limit as shown in the figure below.

This clearly shows that the box is acting at the limit of its capacity as section and if possible, a bigger box should be provided from an efficiency point of view.

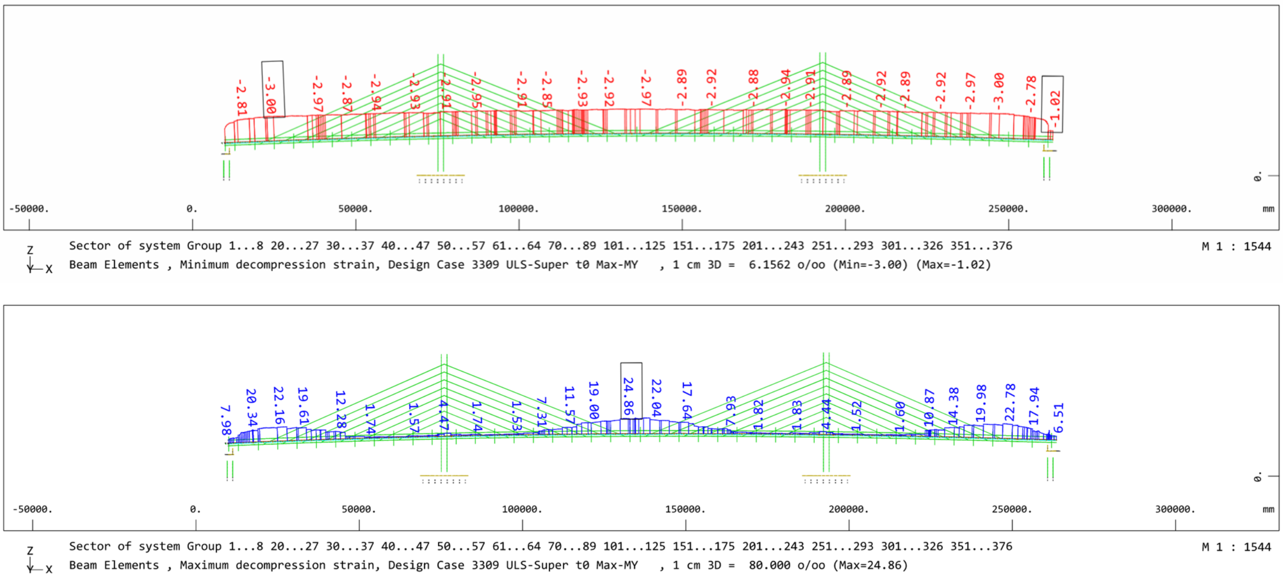

Figure 10. Max concrete strain (top) and prestressing stress strain (bottom) for the My max envelope.

The slab is designed separately, based on the forces obtained in the global model. To optimize the passive reinforcement in the longitudinal direction, the slab is divided in strips, that account for the effect of the shear lag as it is reduced as we move towards the midspan in the transversal direction. Therefore, higher ratios of reinforcement are only provided in the areas with higher stresses (see figures [5] and [6]).

2.3 Superstructure. Other elements

2.3.1 Stay design

As indicated in [1], the stressing sequence of the stays is unconventional as it requires a low stressing force during the cantilever stage and second stressing for every stay. Despite the stress range on the stays due to traffic loading being close to the extrados range (50 to 120 Mpa) the stays have been designed with a limit of 0.45 GUTS, resulting on strands number varying between 40 and 55 between the shortest and the longest stay.

2.3.2 Transverse elements

The transverse T-Roffs both at intermediate locations and at towers were analyzed using the global model and a semi local model replacing the T-Roff beam elements by shell elements at key locations to capture the transversal bending effects.

2.4 Substructure

The integral connection of the deck and the towers makes the bridge behave in the longitudinal direction as a portal frame. For that reason, the relative stiffness of the substructure and superstructure elements strongly influences the results. Upper and lower bound stiffness estimations for the soil are considered, and the bridge components are designed for the envelope of the two. For geometry control, the best estimate of stiffness is used instead.

2.4.1. Calibration of the ground springs

In the global analysis model, the soil is represented by means of linear springs, which vary along the length of the piles. In the horizontal directions, the linear springs for the upper and lower bound geotechnical properties are calibrated using the P-Y curves obtained from the geotechnical analysis and considering the actual displacement of the pile at each level. For each level, the equivalent linear spring representing the force-displacement is obtained, after several iterations.

In the vertical direction, springs characterizing the skin friction are modelled along the length of the pile, as well as a final spring at the pile toe, representing the bearing support. These springs are calibrated to obtain a similar displacement at the pile cap level in the global and geotechnical models.

2.4.2. Accidental scenarios: Earthquake, vessel impact and flood events.

Based on the vessel sizes, frequency and speeds, a risk assessment was undertaken, considering a future ferry as a design vessel, with an equivalent static head-on force of 3.1MN, and an annual probability of collision of 1 in 130,000 years. The impact force has been applied at several locations and heights in the pile caps. Results show that this combination does not govern the pile design over the earthquake and traffic loads.

For the earthquake design, a basic acceleration of 0.09g is anticipated in the area, with a maximum acceleration of 0.56g in the flat area of the bridge design spectrum for periods below 0.4 seconds. A modal-response-spectrum analysis has been undertaken, with ductility factors of 1.0 for abutments and bearings, and 2.0 for towers and foundations. Since the upper part of the piles is not buried, a ductility factor of 2.0 has been agreed with the authority for this bridge.

The most dominant modes are the first two which represent the longitudinal and transversal vibrations of the deck as a portal frame in the short and long directions, mobilizing a significant amount of the total mass in these first modes.

Processing
Eigenvalues

No.	LC	λ [rad2/sec2]	error [-]	ω [rad/sec]	f [Hz]	T [sec]	ξ [%]	Meff X[%]	Meff Y[%]	Meff Z[%]	participation X[%]	participation Y[%]	participation Z[%]
1	9101	1.2716E+01	-	3.566	0.568	1.762	5.000	76.3	0.0	0.0	76.3	0.0	0.0
2	9102	1.8853E+01	-	4.342	0.691	1.447	5.000	0.0	75.6	0.0	0.0	75.4	0.0
3	9103	1.9301E+01	-	4.393	0.699	1.430	5.000	0.3	0.0	6.5	0.3	0.0	6.5
4	9104	5.1284E+01	-	7.161	1.140	0.877	5.000	0.0	0.1	0.0	0.0	0.1	0.0
5	9105	6.2319E+01	-	7.894	1.256	0.796	5.000	1.7	0.0	4.3	1.7	0.0	4.3
6	9106	6.8084E+01	-	8.251	1.313	0.761	5.000	0.0	0.1	0.0	0.0	0.0	0.0
7	9107	8.3887E+01	-	9.159	1.458	0.686	5.000	0.1	0.0	36.4	0.1	0.0	36.4
8	9108	1.0858E+02	-	10.420	1.658	0.603	5.000	0.0	0.0	0.1	0.0	0.0	0.1
9	9109	1.1022E+02	-	10.499	1.671	0.598	5.000	0.0	0.0	0.4	0.0	0.0	0.4
10	9110	1.2183E+02	-	11.038	1.757	0.569	5.000	0.0	0.3	0.0	0.0	0.4	0.0

Figure 11. First longitudinal mode and mass participation of the first ten modes

The structure has demonstrated a strong influence of the geotechnical parameters on the overall behaviour, with the first longitudinal vibration mode ranging from 0.75 Hz to 0.56 Hz for the upper and lower bound ground stiffness respectively, resulting in very different forces longitudinally and transversally.

Reduced stiffness due to concrete cracking is also implemented in the analysis, ranging from 50 to 60% of the gross properties of the sections.

2.4.3. Uneven ground conditions at Tower 2

The ground investigation at Tower 2 location consists of two boreholes per pile group. The four boreholes, along with other nearby boreholes were analysed, revealing a high variability in the Leached Tamala Limestone, as shown in figure 12 below.

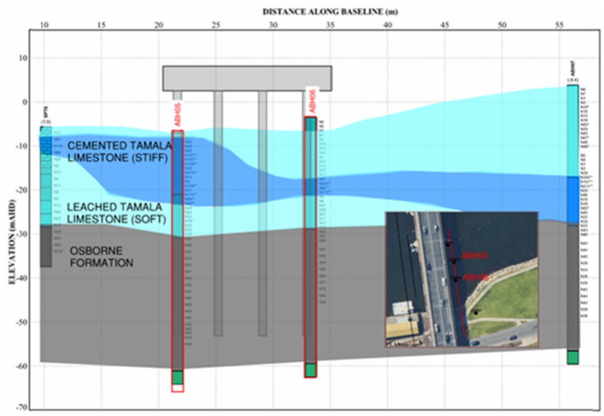

Figure 12. Geotechnical profile at Tower 2 East

To prevent different piles on the same pile cap from having very different fixed lengths which will result in a concentration of bending on those piles reaching the hard layers first, an external liner ring consists of a steel ring welded to the outside of the sacrificial liner for approximately 14m has been designed for all piles in Tower 2.

3. Local Models

Several local models have been developed as part of the detailed design analysis. This section summarizes some of the models used.

3.1 Pile caps

The pile cap for each tower has an irregular hexahedral shape due to the architectural requirements and the 4+3 pile configuration. Two different local models were use: for ULS, a strut-and-tie model was developed, using the software MIDAS. At pile locations, the stiffness of the supports was calibrated with the global model to obtain similar reactions.

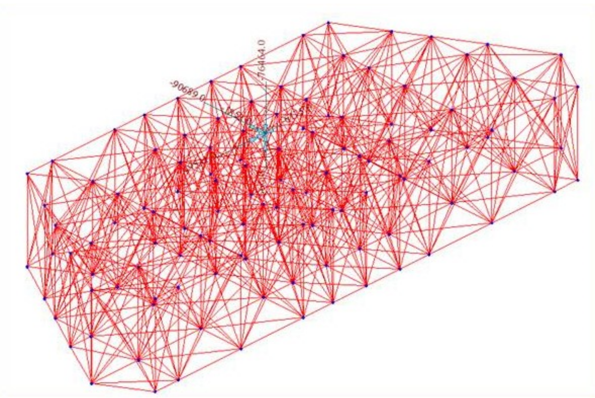

Figure 13. Strut and tie model for the pile cap design

With the reinforcement obtained from the strut-and-tie model, a non-linear 3D volumetric model was developed using the software LUSAS including the reinforcement and non-linear properties in the concrete. The concrete and reinforcement stresses obtained with this model at sections close to the tower base were integrated to obtain an equivalent Axial and bending moment and crack width checks were carried out with these values.

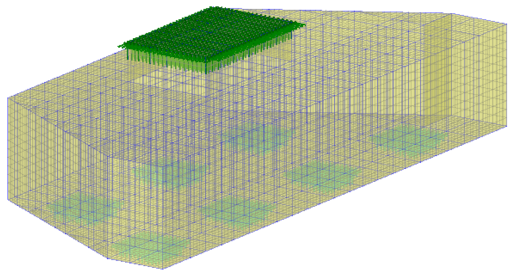

Figure 14. 3D volumetric model for the pile cap

3.2 Cable anchors at deck level

The cable-stays are anchored to the deck at the top of the edge girder segments. An anchor block with a variable shape, approximately 1m deep at its maximum and with the full width of the narrow segment, is provided at the top of the segment. A cast-in steel guide pipe where the cable is installed and the anchor block itself, stand out from the top flange of the edge girder.

The figure 15 below illustrates the Strand7 model for the segment that includes the anchor of the stay-cable. The model comprises of 3D brick elements (Tetra10) and the steel collar has been modelled using plate elements (Tri6).

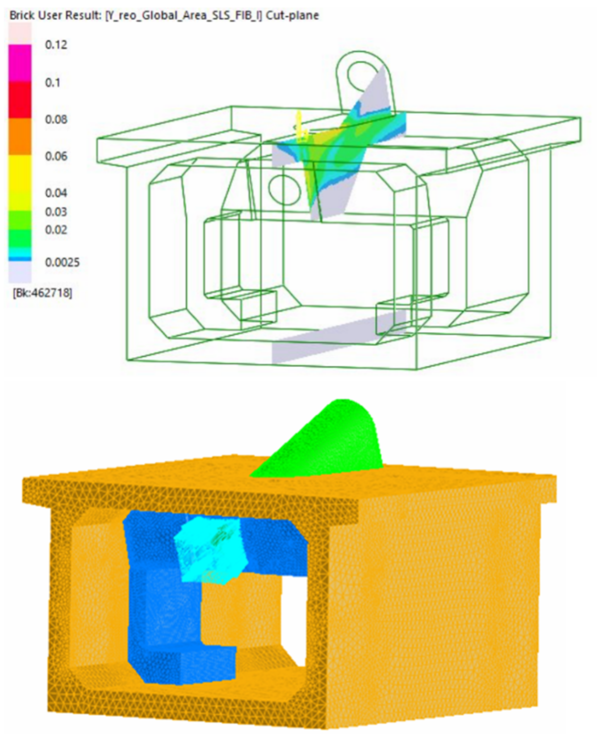

Figure 15 Cable-stay anchor, modelled with Strand7

3.3 Deck Tower Nodal Zone

The nodal zone of the deck and the tower was used considering 2D Strut and Tie models on each plane. This area is extremely complex as it also includes the anchor of the longitudinal PT and a transverse beam connection.

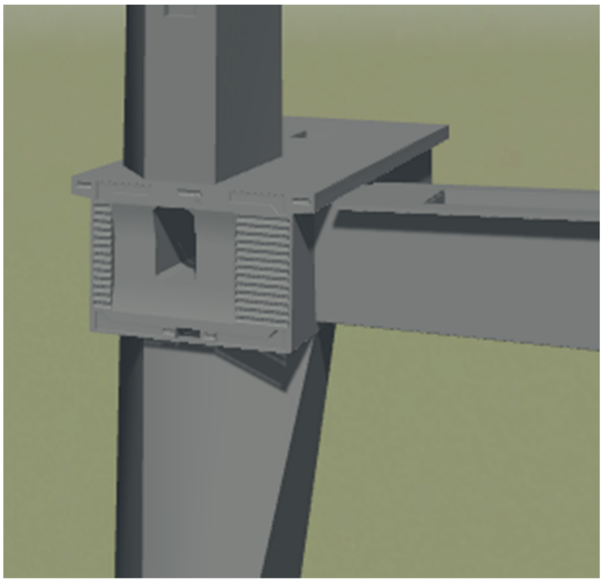

Figure 16 Nodal Zone, 3D drawing model

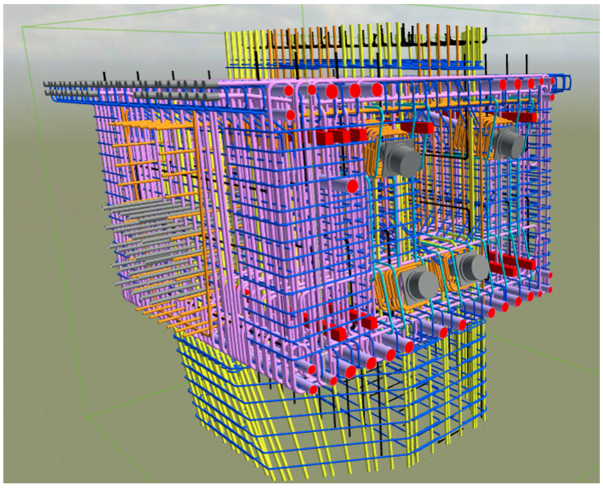

Figure 17 Nodal Zone, 3D reinforcement model

3.4 Deck to T-Roff Connection

Similarly to the stay anchor a local model using Strand 7 has been developed for this area. The figure below shows a results output of the model.

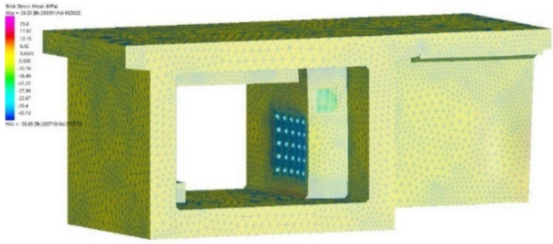

Figure 18. Analysis model for the Edge girder – transverse beam connection

4. Construction sequence

The construction sequence is highly influenced by the requirement that the old timber bridge, which is in the same footprint, shall remain open for as long as possible, to minimize traffic disruption. The foundations, towers and edge girders are built using a balanced cantilever method, on both sides of the existing bridge, while the latter is still functional.

The existing bridge is then dismantled prior to the installation of the transverse beams and slab.

The 19 precast segments of each cantilever are installed by crane lifting, using the temporary platforms already installed in the river at both sides of the existing bridge for the foundation construction. The typical cantilever cycle includes the installation of two segments and a cable-stay. Segments are attached to the previous ones by means of temporary 40mm diameter stress bars.

Symmetrically arranged from segments 2, 7 and 13, a pair of top and bottom permanent tendons are installed and grouted, so that the full cantilever does not have to rely only on external temporary stress bars for the full length.

Provision for the correction of construction tolerances in the cantilevers at closure of 100mm in vertical and transversal directions, in addition to the horizontal jacking described in the Conceptual design paper (see [1]) have been taken into account.

Figure 19. Installation of piles near the existing bridge, from temporary platform.

The second stressing of the cable-stays and the introduction of the continuity PT of the boxes occurs in different stages. The shorter ones are stressed once the edge girders are closed, and the internal prestressing installed, whereas the longer ones either after installing the transverse beams, between slab pours, or after casting the full slab. Permanent formwork panels are used to cast the slab.

At the time of writing this paper, the piles of the main towers were under construction as well as the precast segments, with the erection of the edge girders currently expected between July and November 2025 and the bridge to open 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.

Also, the wider design team, which includes dozens of staff from ARUP and WSP, and finally the other 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.

References

[1] M. Sanchez, Et al., Conceptual design of the new bridge in Walyalup (Fremantle), the first precast segmental cable stayed bridge in western Australia, Congreso de la Asociación Científico-Técnica del Hormigón Estructural (ACHE) 2025.

[2] Standards Australia (2017) Bridge Design Part 2: Design Loads, AS 5100.2:2017.

[3] Standards Australia (2017) Bridge Design Part 5: Concrete, AS 5100.5:2017.

[4]AASTHO LRFD Bridge Design Specification.Section 5.12.4.3.2 Design of Segmental bridges. Construction Loads