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Structural Design of the New Bridgewater Bridge in Hobart, Tasmania

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The New Bridgewater Bridge (NBB) Project is Tasmania's largest ever transport infrastructure project. The project is a critical link in Tasmania's transport network and forms part of the Australian Government's National Land Transport Network. The NBB project involves construction of a four-lane, 1.28 kilometre long bridge over the River Derwent with new interchanges at Granton and Bridgewater, a shared path for cyclists and pedestrians, and a navigation clearance consistent with the Bowen Bridge, all aimed at improving traffic flow and connectivity. The NBB replaces the existing steel-truss vertical lift bridge, which was completed in 1946.

The bridge consists of a twin box girder supported on twin piers and monopiles. The bridge deck was precast off-site in segments and erected by a balanced cantilever method using internal post-tensioning with span continuity provided by external tendons.

The structural design was developed to maximise construction efficiency by limiting the number of very long piles to be constructed by using a monopile system, developing an innovative method of constructing the joint between the piers and piles, limiting precast segment weights to reduce the size of cranes needed for their handling and optimising the reinforcement in both segments and piles to minimise assembly times.

This construction driven design required a high degree of collaboration between designer, contractor, proof engineer and independent verification engineer in order that any potential construction difficulties could be eliminated by design and that approval of the designs could be obtained and construction could proceed in a timely fashion.

Keywords: Bridge design, precast segmental, balanced cantilever, prestressing, monopile

1. Introduction

Tony Gee and Partners were engaged by McConnell Dowell Constructors (MCD) in 2021 to prepare a tender design for the proposed River Derwent Bridge, a new crossing of the River Derwent forming a part of the New Bridgewater Bridge Project (NBB). The project included construction of a new four lane bridge (dual two-lane carriageways) downstream of the existing Bridgewater Bridge, a shared user path for pedestrians and cyclists, as well as an upgraded intersection at Granton to provide direct access between the Brooker and Lyell Highways and direct access from Bridgewater to the new bridge.

The project will fix the missing link in the Tasmania National Highway network, benefiting the 22,000 people that will cross the bridge daily, as well as providing an un-interrupted route for traffic between Hobart and New Norfolk and improving the links for local traffic.

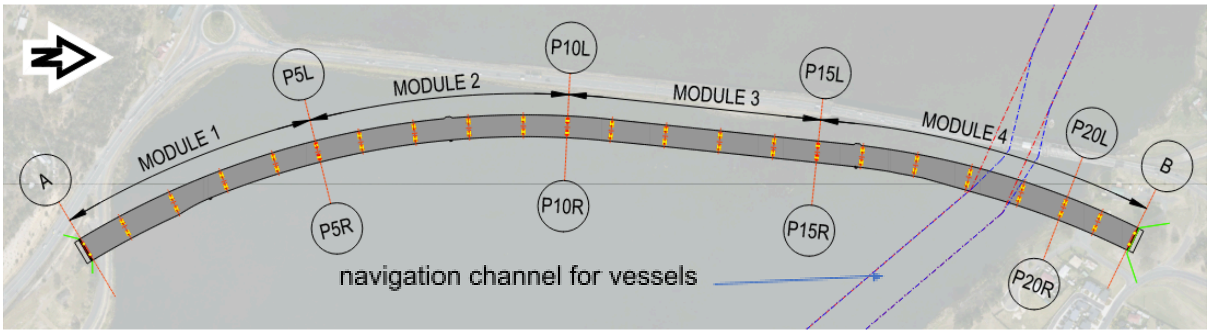
The project was procured via a comprehensive Early Contractor Involvement (ECI) tender process where two tenderers were selected to develop designs for the project, working collaboratively with the Department for State Growth. MCD's tender was ultimately successful, winning the Design and Construct contract, and Tony Gee were subsequently commissioned to prepare the detailed design of the bridge.

2. Layout of the Bridge

General Layout

The bridge structure over the River Derwent is 1271.750m long and starts just south of Main Road, Granton adjacent to the existing Brooker Highway and ends just north of Gunn Street, Bridgewater. The bridge consists of 22 spans and is split up into 4 continuous bridge modules in a 5-5-5-7 number span configuration with a typical span of 64m. A navigation channel is provided between Piers 18 and 19 in module 4, providing a navigation clearance consistent with that of the Bowen Bridge, located 11 kilometres downstream.

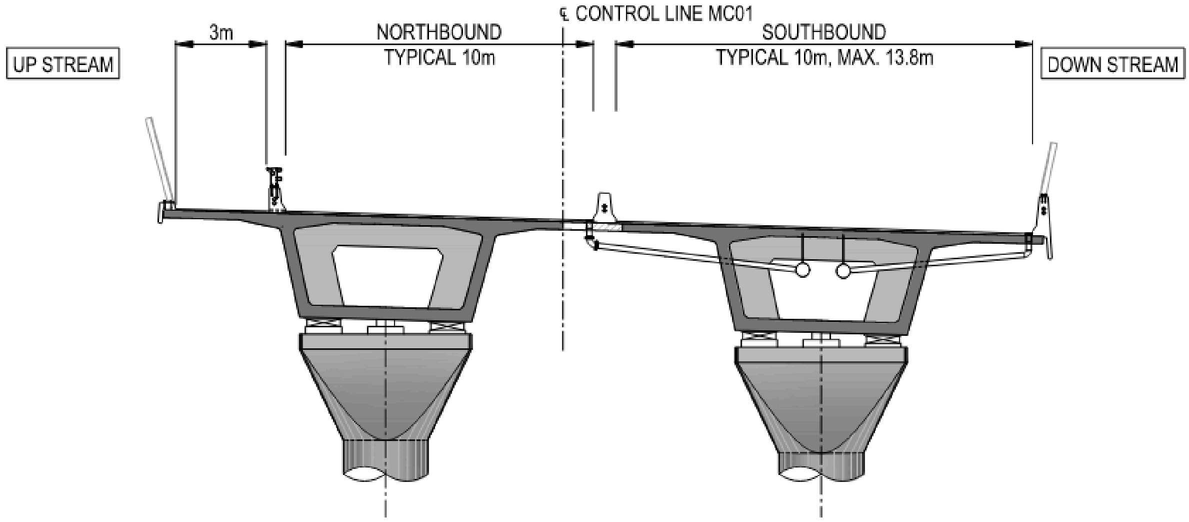
Figure 1 - Plan layout of bridge



The bridge deck varies in width being typically 25.15m wide, to accommodate two lanes of traffic with 2.0m outside shoulders in each direction along the centre sections, but flares to 29.15m wide for an additional lane to allow for the Southbound exit and entry ramp merges. There is also a 3.00m wide shared user path (SUP) along the western side of the bridge deck. The bridge has a constant 2.2% crossfall on the SUP and 3% cross fall on carriageway towards the eastern side of the bridge.

The edge pedestrian barrier of the SUP is combined with a 3m high safety screen to work as an integrated assembly. A safety screen is fixed on top of the 1.2m high medium performance level edge traffic barrier for the southbound carriageway. The safety screen angles are designed to suit vehicle rollover, sign gantries, and for pedestrian safety. The bridge supports two ITS gantries and other sign gantries as required. The typical cross section of the bridge is shown in figure 2 below.

Figure 2 – Typical cross sections of bridge



The bridge was designed to allow for the future installation of an underslung services gantry underneath the soffit in the centre of the bridge. The gantry will accommodate 1 no 400mm diameter sewer main, 2 nos 300mm diameter water mains and a 1.2m wide maintenance accessway.

Bridge Superstructure

The bridge superstructure has the structural form of twin box girders of constant depth, prestressed precast segmental concrete. Each single cell box girder is used to support each side of the carriageway. The box girder is trapezoidal with the webs sloped at approximately 14 degrees to the vertical. The overall depth of the box is 3.4m and the segment soffit is a constant 5.7m wide. This constant shape maximises the benefits of the precast construction method. The increase in deck width at the ends of the bridge was accommodated by increasing the length of the deck cantilevers with the width between the centreline of the deck boxes varying to accommodate the varying road width.

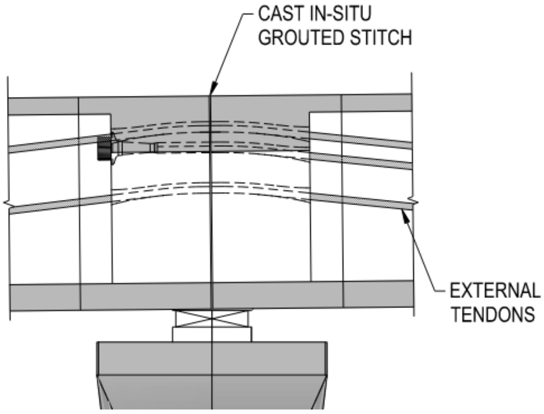
The adjacent box girders are designed to be connected together to provide a continuous top slab for the 75mm road surfacing. For this purpose, a longitudinal reinforced concrete cast in-situ concrete stitch, nominally 1.2m wide, was designed to be cast after erection of the box girders to connect their inner cantilevers together.

The deck cantilevers were locally extended at the locations of the ITS and sign gantries in order to provide support to the gantry legs and to make the gantry leg support independent of the bridge parapets.

The box girders are designed to be constructed using the match-cast segmental method using segments that were precast off site and erected by balanced cantilevers. The segments are longitudinally prestressed using a combination of internal and external prestressing. The internal prestressing cantilever tendons consist of 11 or 12 number of 15.7mm nominal diameter strands (with plastic ducts and anchorage units), with each pair of tendons anchored between the leading edges of the segments in the cantilever under construction. The continuity tendons are located within the boxes comprising of 31 number 15.7mm nominal diameter strands (encased in HDPE ducts and filled with grout) as external tendons of 1 to 3 span lengths and are anchored in the pier diaphragms and the first blister in the segments in the quarter span region. No transverse prestressing in the segment top slab is provided, with the required strength being provided by standard transverse reinforcement.

There are two pier segments at each internal pier, so detailed in order to fulfill an operational requirement of 90T maximum lifting weight. Each pier segment is therefore 1.99m long and a wet-grouted joint of 20mm nominal thickness was provided between the two pier segments. Refer to figure 3 below. The thickness of the diaphragm internal walls is 1.5m for each pier segment (in the longitudinal direction). When adjacent pier segments are jointed, these diaphragm walls are used to anchor the external tendons and to accommodate the bearing forces from both the permanent bearings and also the temporary jacking points used during construction.

Figure 3 – Typical longitudinal cross section at piers showing twin pier segments



Each expansion joint pier segment is 2m long and has a 1.5m long internal diaphragm wall. The weight of this pier segment is around 84 tonnes. The diaphragm wall of this segment is used to anchor the 2 x 4 No. 6-31 external tendons as well as accommodating the bearing forces from both the permanent bearings and also the temporary jacking points used during construction

The concept behind the bearing layout design was that each pier was to be pinned to each span in order to distribute longitudinal loads as evenly as possible between all the foundations. All piers are thus pinned to the deck except at expansion joint piers, where one module is fixed to the pier and the adjacent module is not. In this manner all longitudinal loading on the bridge is shared by all the piers, resulting in an efficient distribution of load and a cost-effective substructure design.

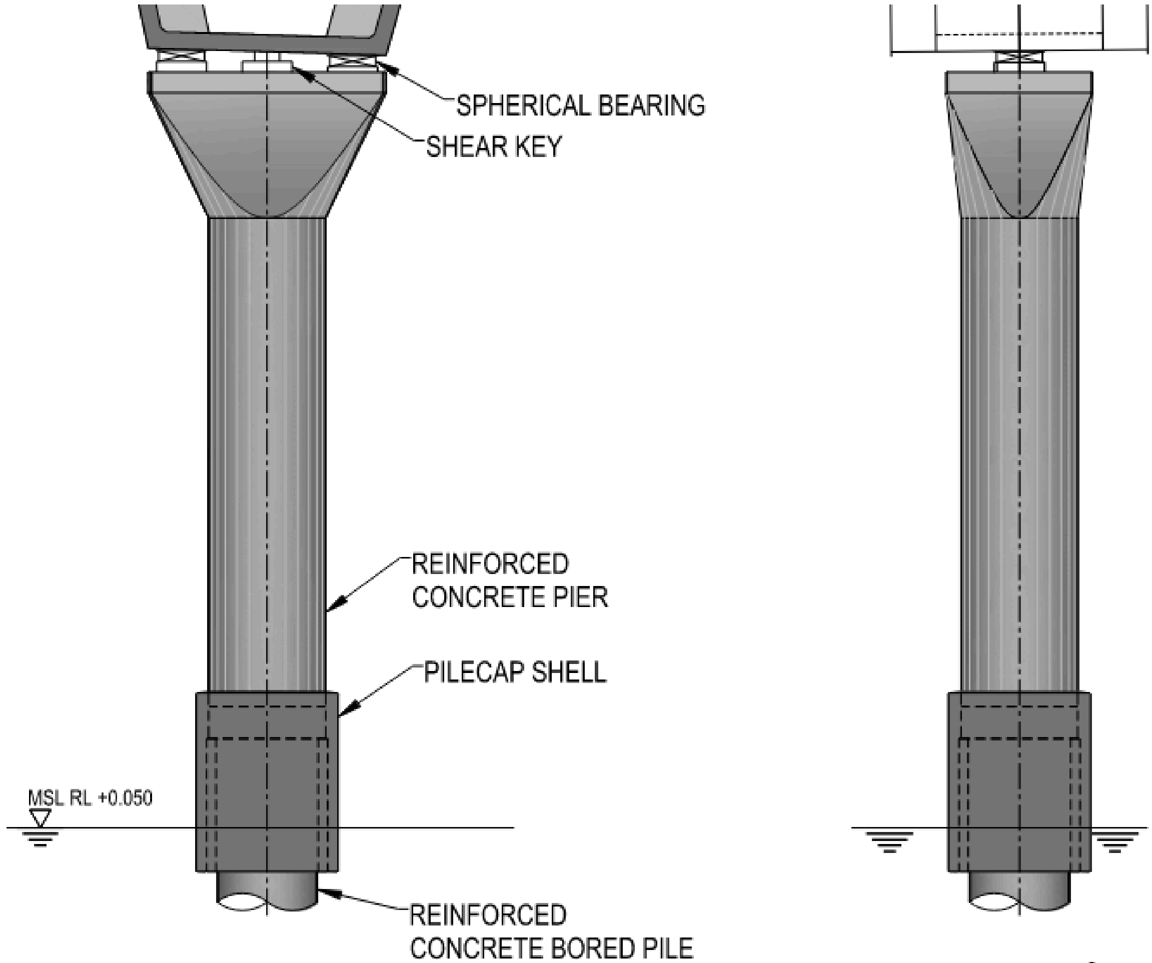
At the typical internal pier there are two free sliding bearings, one underneath each web of the box girder and one centrally located fixed shear key. Refer to figure 4 below. At the expansion joint pier, the deck of one module is supported by two free sliding bearings with one fixed shear key, and the deck on the other module is supported by two free sliding bearings with one sliding shear key aligned longitudinally.

At the North and South abutments, the two box girders are connected together by a 1.2m wide in-situ diaphragm and two bearing provided underneath each box girder, with a centrally located shear key to resist transverse loadings.

Bridge Substructure

At every pier each box girder is supported by a single reinforced concrete column and a single large diameter reinforced concrete bored pile as shown in figure 4 below. The columns are 2.8m diameter and are cast on top of a 2.45m diameter pile which is socketed into the underlying rock to form monopile foundations.

Figure 4 - Pier monopile layout



There are some marine deposits in the underlying strata so the piles are lined with a sacrificial steel casing from the top of the pile at +2.2m AHD down to such a level that the concrete of the bored pile can be unsupported during the concreting stage. The piles are socketed into the underlying rock to provide the required structural fixity. With the increase in pile casing thickness at its shoe to 40mm and allowing for a tolerance of 20mm between the casing and the rock socket drill bit for insertion of the latter, the diameter of the pile (structural concrete diameter) at the rock socket was necessarily reduced to 2.38m.

The pier columns are 2.8m in diameter and vary in height between 7.7m and 15.8m. The top of the pier flares both longitudinally and transversely to provide a pier top area of 5.7m x 3.5m, which provides sufficient space for the bearing plinths and all of the temporary works required for erection.

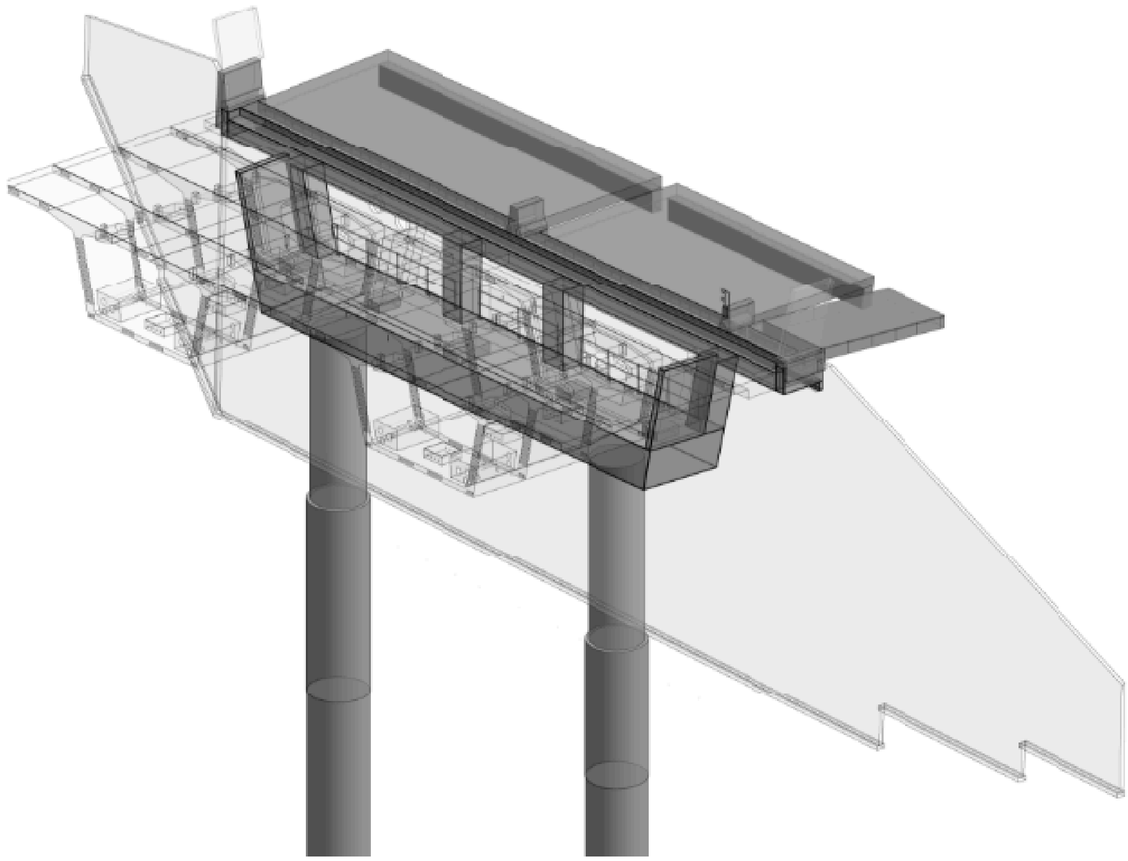
The pier is connected to the pile with a precast pilecap annulus shaped shell that slots over the top of the pile, whose top level is +2.2m AHD. The shell annulus extends up outside the pier column to a level of +3.3m AHD, fully encasing the construction joint between pier and pile. In this manner the joint is protected from the effects of the tidal splash zone, therefore enhancing the durability of the structure.

Bridge Abutments

At the North and South abutments, each box girder is supported by 2.14m diameter bored piles and 1.88m diameter column extensions. There is a rectangular cross beam connecting to the top of the two pier columns that acts as the abutment bearing shelf. The box girders are connected together at these locations with a full depth structural diaphragm. An image of the typical abutment is shown in figure 5 below.

There are reinforced soil structure (RSS) retaining walls located behind and independent of the abutment support beams, so the bridge abutments are “expressed” from the walls. A chamber is provided between the retaining walls and the box girders in order to allow access into the box girders.

Figure 5 - Southern abutment



Traffic Barriers

Medium performance level barriers are provided to both sides of the carriageway along the entire length of the crossing. On the northbound edge, the barrier consists of a 750mm high concrete F-shape barrier and topped with a twin steel rail to a combined height of 1.4m measured from the top of surfacing. On the southbound edge, the barrier consists of a 1.2m high concrete F-shape barrier and topped with a safety screen to a combined height of 3.0m from the top of surfacing. For the median barrier that separates the carriageways, a regular performance level barrier is provided. This barrier consists of a 920mm high concrete F-shape barrier only. Vertical barrier joints are provided at regular intervals to prevent undesirable cracking in the parapets, particularly over the piers in regions of negative bending.

On the SUP edge, a concrete kerb has been provided for the 3.0m high safety screen.

3. Key features of the Structural Design

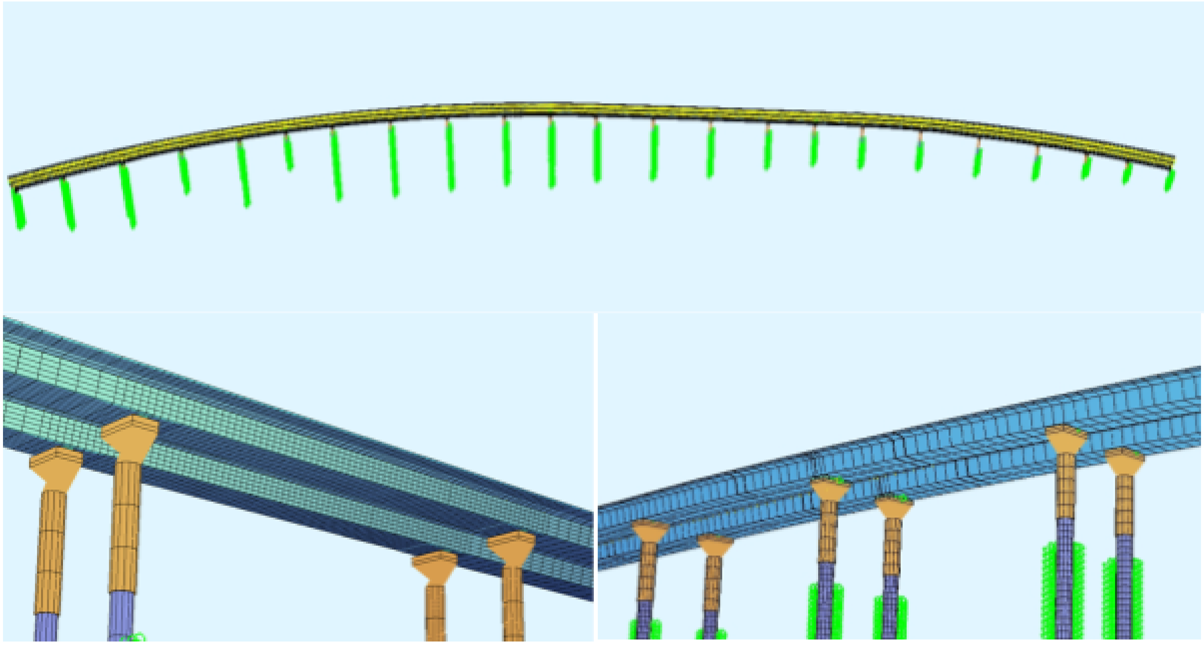
The design of the bridge required some complex structural analysis, the key features of which are described as follows:

Structural Modelling

To determine the resulting design actions for the structure during construction, as well as in its final service configuration, a number of structural analysis models were used. The main models include those listed below and illustrated in figure 5 below:

- A 3D beam type global model of the structure including piers and piles in SOFiSTiK 2020.
- A 3D finite element plate model of the deck box girder in SOFiSTiK 2020.
- 3D finite element brick models for detailed reinforcement design of individual structural elements such as diaphragm, deviators and anchorages.

Figure 6 - Structural analysis models



The construction stage analysis, including staged application of post-tensioning, together with time related effects are accounted for in the long-term dead load actions using the SOFiSTiK global model to determine the redistribution effect from time dependent material effects (concrete creep and shrinkage only) as well as changes in support and restraint conditions, including any accumulated locked-in force effects resulting from the construction process. The design of the transverse actions induced on the deck section were determined from the 3D plate model, which included the effect of torsional and distortional warping.

Monopile Design

Monopiles rely on the lateral support they receive from the surrounding soil and rock for their stability. This lateral support from the soil and rock keeps the pile in its position under the influence of lateral loadings, an effect commonly known as soil structure interaction. In areas where the rock is close to the surface, the piles are effectively socketed into this rock and the piles act as simple cantilevers under the influence of lateral loadings.

However, in some other areas along the length of the bridge the rock is quite deep with resulting pile lengths of up to 90m, so the stability of the pile is provided by the soil above the rock. Here the stresses and movements of the pile and the soil influence each other and depending upon the soil type, this relationship can be non-linear when the passive loading limit of the soil is reached. Complex structural and geotechnical analysis was required to accurately predict the behaviour of the monopiles and consideration had to be given to the likely variation in ground stiffness compared to the results of the site investigation.

Both lower bound (50% of mean) and upper bound (200% of mean) ground stiffness were considered in the design for the normal static load combinations. Under dynamic loading scenarios such as impact and seismic loads, dynamic spring stiffness were adopted. Effect of scouring was considered by removal of horizontal soil springs that are within the scouring depth in the global analysis

Of particular concern to the structural engineer is the buckling behaviour of the piles, because the resistance to buckling is provided by the lateral support of the layers of soil. In this instance some of the upper soil layers are quite weak and susceptible to scour during river flooding events, so with the removal of layers of soil the buckling resistance of the piles will reduce.

Large diameter piles such as the ones used on this project usually have buckling capacities far in excess of the loading that could ever be applied to this bridge. But in this instance some of the main river piles had to be considered to be unsupported for a considerable length, combined with the tall height of these piers at the crest of the vertical curve of the bridge, meaning that the pier / pile monopile combination had to be designed with unsupported lengths of up to 33m.

The AS5100 bridge design code approaches this design situation using the "moment magnifier" method, which calculates the additional moments to be used in the structural design of the pile on top of those created by the applied loadings.

The calculation of moment magnifier d_b for a braced column is specified in Cl. 10.4.2 of AS5100.5[1], reproduced below:

The moment magnifier (δ) for a braced column shall be taken to be equal to δ_b given by—

$$\delta_b = k_m/(1-N^*/N_c) \geq 1 \qquad \ldots 10.4.2$$

where

N_c = buckling load given in Clause 10.4.4

k_m = $\left(0.6 - 0.4M_1^* / M_2^*\right)$ but shall be taken as not less than 0.4, except that if the column is subjected to significant transverse loading between its ends and in the absence of more exact calculations, k_m shall be taken as 1.0

The ratio of M1*/M2* is defined in Cl. 10.3.1, reproduced below:

M_1^* / M_2^* = ratio of the smaller to the larger of the design bending moments at the ends of the column

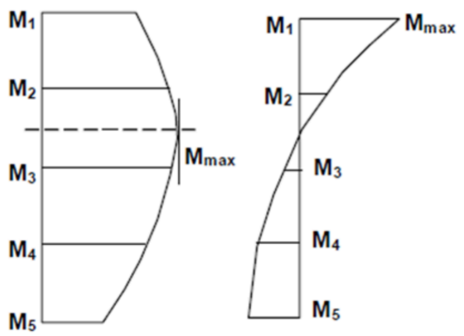
The ratio shall be taken as negative when the column is bent in single curvature and positive when the column is bent in double curvature. When the absolute value of M_2^* is less than or equal to $0.05DN^*$, the ratio shall be taken as -1.0

k_m is the equivalent uniform moment factor, which is used to cater for different loading and end restraint conditions in calculating d_b. For the typical load case at which the column and pile is not subjected to significant transverse loading between its ends, k_m is calculated based $0.6\text{-}0.4M_1^* / M_2^*$ as given in AS5100.5.

For the load case at which the column and pile is subjected to horizontal loading between its ends, such as the case of vessel impact at water level and collision from road traffic at ground level, AS5100.5 specifies that k_m shall be taken as 1.0 "in the absence of more exact calculations". It is very conservative to take k_m as 1.0, an approach which completely ignores the bending moment profile of the member. More exact calculations are provided in AS5100.6[2], where guidance is given for the case of members with horizontal loads applied between two ends, but this is only for the case where the loads are applied at the midpoint of the member. This was not the case for the Bridgewater Bridge piers, where ship vessel impact occurs elsewhere in the body of the pier/pile combination.

In order to not unduly penalise the design, reference for k_m was made to the Hong Kong Codes of Practice[3] where the bending moment profile of the member can be considered. Figure 7 below shows the formula for calculation of the additional moment factor m, being equivalent to k_m. It should be noted that for the case where a point load is applied at the midpoint of the member this HK approach is more conservative than the method in AS5100.6.

Figure 7 – Calculation for additional moment factor m, extracted from Hong Kong codes of practice.



$$m = 0.2 + \frac{0.1M_2 + 0.6M_3 + 0.1M_4}{M_{max}} \text{ but } m \geq \frac{0.8M_{24}}{M_{max}}$$

The method of analysis and design was therefore as follows:

- 3D structural analysis using gross section properties for all concrete structural elements
- Geometric non-linearity included in the structural analysis
- Material non-linearity for the supporting soil springs included in the structural analysis
- Evaluate the moment magnifier according to AS5100.5 clause 10.4.4 with k_m in accordance with the HK Codes of Practice.

This approach was verified using a rigorous approach with the full geometrical non-linear frame analysis specified in AS5100.5 clause 10.2.3 and complying with the requirements of non-linear frame analysis given in clause 6.5. This approach used the following:

- 3d structural analysis using cracked stiffness applied to piers and piles
- Gross section properties applied to prestressed deck
- Geometric non-linearity included in the structural analysis
- Material non-linearity for the supporting soil springs included in the structural analysis

Using this design method the peak design bending moment in the tallest piles was calculated to be 46.2 MNm under the ship impact case whereas using the rigorous approach the moment reduced to 41.5 MNm, thus allowing a margin of some 10% in the design.

Construction Programme Modelling

As discussed above, the design allowed for the staged construction method of the bridge. During the construction phase this structural model was expanded to represent the detailed construction activities and staging so that the impact of any required changes to the construction programme could be speedily assessed. An example of this was the timing and sequence of the installation of the bridge deck barriers relative to the sequence of casting the midspan cantilever stitches and stressing of the continuity tendons. The step-by-step structural model could assess the impact of any sequence of installation and together with MCD, the construction sequence was optimised to reduce the construction period.

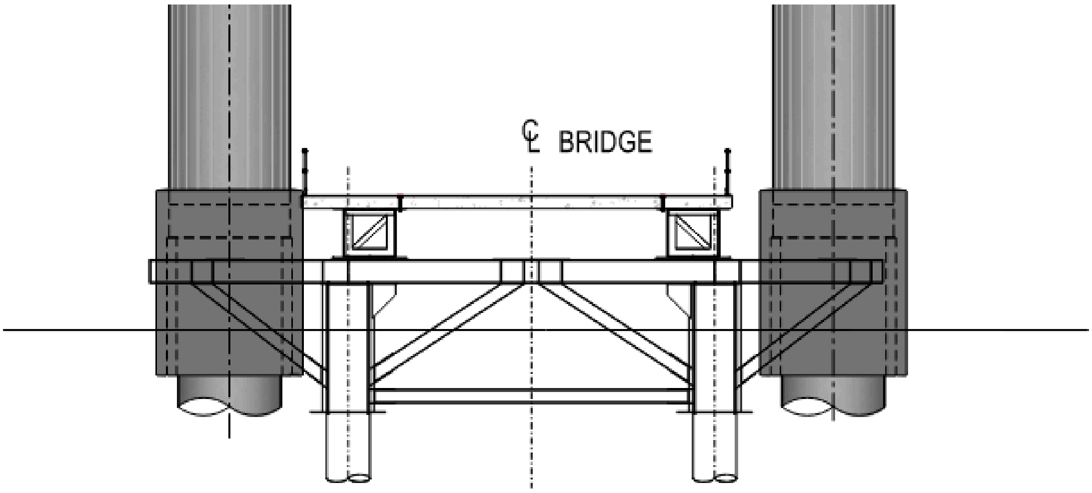
4. Design for Construction

In a design and construct contract such as this, consideration of how the design is to be constructed is always at the forefront of the designer’s mind. Several key issues that drove the design of the bridge are explained as follows:

Cross Section Dimensioning

The twin box layout of the bridge was chosen specifically to suit the layout of the temporary bridge that would be needed for the construction of the marine bridge. The temporary bridge was planned to run down the middle of the 25m wide main bridge, so that access for the monopile and pier construction could be made from each of its sides, rather than using a temporary bridge to one side of the main bridge with longer side fingers out to the pier positions. The layout of the temporary bridge is shown in figure 8 below. The central temporary bridge also suited the planned method of erection using a twin lifting frame that could lift up segments from the middle of the bridge and slide them laterally into their final position.

Figure 8 - Relationship between temporary bridge and piers



The centrelines of the box girders were therefore positioned a constant transverse distance apart of 14.00m for the 25.15m wide portion of the deck and 14.43m apart for the 29.15m wider section of the deck at each pier. The 10m wide temporary bridge was aligned between the piers and below the inner cantilevers of the box girders.

Joint between Pile and Pier

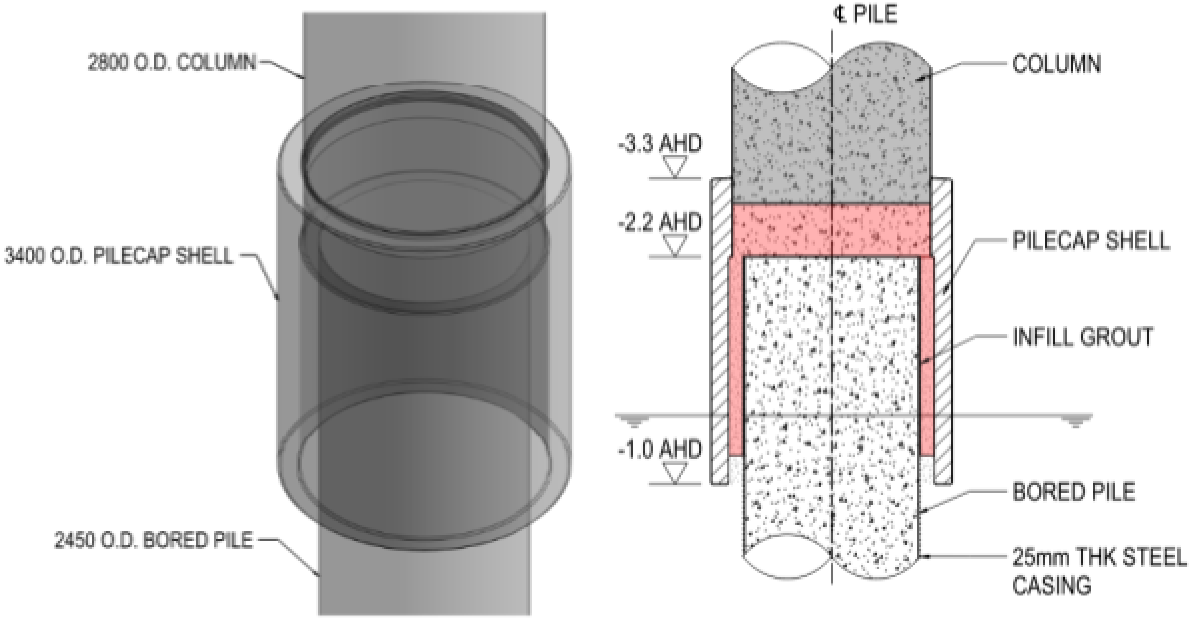
Use of monopiles can sometimes create a construction issue because piles are not always constructed in the correct place. This is recognised in the project specifications, which allowed a 75mm positional tolerance for piles, compared to a much tighter positional tolerance for other elements, such as the piers. So, when a design uses monopiles, the design must allow for the inevitable geometrical mismatches that will occur between pier and pile, which would otherwise cause difficulties for the continuity of reinforcement between pier and pile.

In this instance, the diameter of the pier was fixed at 2.8m in order to be larger than the 2.45m diameter of the pile, so that any tolerance in the plan location of the pile would not affect the positioning of the pier. The monopiles were designed for a 75mm plan pile position tolerance at the top of pile and a 1 in 100 variation from the vertical, i.e. an inclination of the pile, but in theory the dimensioning allowed for the contingency of the pile head being out of position by up to 160mm. However, it is worth noting that such a contingency was never required on this project.

Traditionally, for a marine structure, the construction joint between pier and pilecap is placed below the lowest tide level, so that the joint is out of the tidal zone where the worst corrosive conditions exist. For the River Derwent at this location, the shallow mud flats which exist for more than two-thirds of the crossing would have resulted in expensive temporary works to excavate down to this level. It was therefore the preference of MCD that this joint was to be located above the existing river level, so that this work could be done “in the dry” without such temporary works.

This meant the construction joint was located at +2.2m AHD, well above the river’s highest astronomical tide level of +0.96m AHD. If no other measures had been taken with the joint at this level, then the top of the pile would have been visible with the permanent steel casing exposed in the tidal zone and subjected to unsightly corrosion and contamination of the river. Therefore, a precast concrete annulus was designed to slot over the top of the pile and extended down to a level of -1m AHD, just below the level of the lowest astronomical tide of -0.83m AHD. These relative levels are shown in figure 9.

Figure 9 - Precast pile annulus shell



The shell was concreted to the top of the pile using a 750mm thick slab that was poured in-situ, with the pile reinforcement passing through and up into the main body of the pier. The gap below this slab between the annulus and pile, notionally 160mm wide but varied depending upon the final position of the pile, was filled with concrete to protect the steel casing of the pile from corroding in the tidal zone.

The pier column was then constructed inside the annulus which itself extended up to a level of +3.3m AHD. This meant that the joint between pile and pilecap slab and pilecap slab and pier were fully protected by the walls of the annulus and not exposed to any corrosive conditions. The pile reinforcement extends up into the pier column and the lap between the pier and pile reinforcement occurs in the base of the pier column above the pilecap slab.

Segment length and weight

The success of any segment precasting operation is largely governed by the repeatability of the shape of the segments. A project that has 1000 segments that are each identical will be easier and faster to construct than a project with 1000 segments that are of varying dimensions. However, achieving such standardisation is often very hard due to functional requirements such as the varying road widths and operational restrictions on the weight of the segments for handling and erection. In this instance, the length of the box girder precast segments was determined by the maximum weight that could be practically handled for the delivery and lifting and erection of the segments. These weight limits were set by MCD to be 60 tonnes for erection of typical segments using their lifting frame and 90 tonnes for erection of the pier segments by crane.

These constraints meant that typical segments could only be a maximum of 2.6m long. Segments that contained concrete deviator blocks for the external tendons were reduced in length to 2.2m, because of the weight of these blocks inside the segments. The pier segments were reduced further to 2m in length as their internal diaphragm walls contributed significantly to the overall weight of the segment.

The varying curvature of the bridge alignment also impacted the segment lengths, as did the varying carriageway widths. Consequently, the segment lengths varied between 2m and 2.6m, which was not ideal as it meant that each segment cast was typically a different length to the previous one, introducing complexity in the precasting process. With the benefit of hindsight, a more optimised solution for precasting may have been able to be achieved if the maximum segment lifting weights were increased.

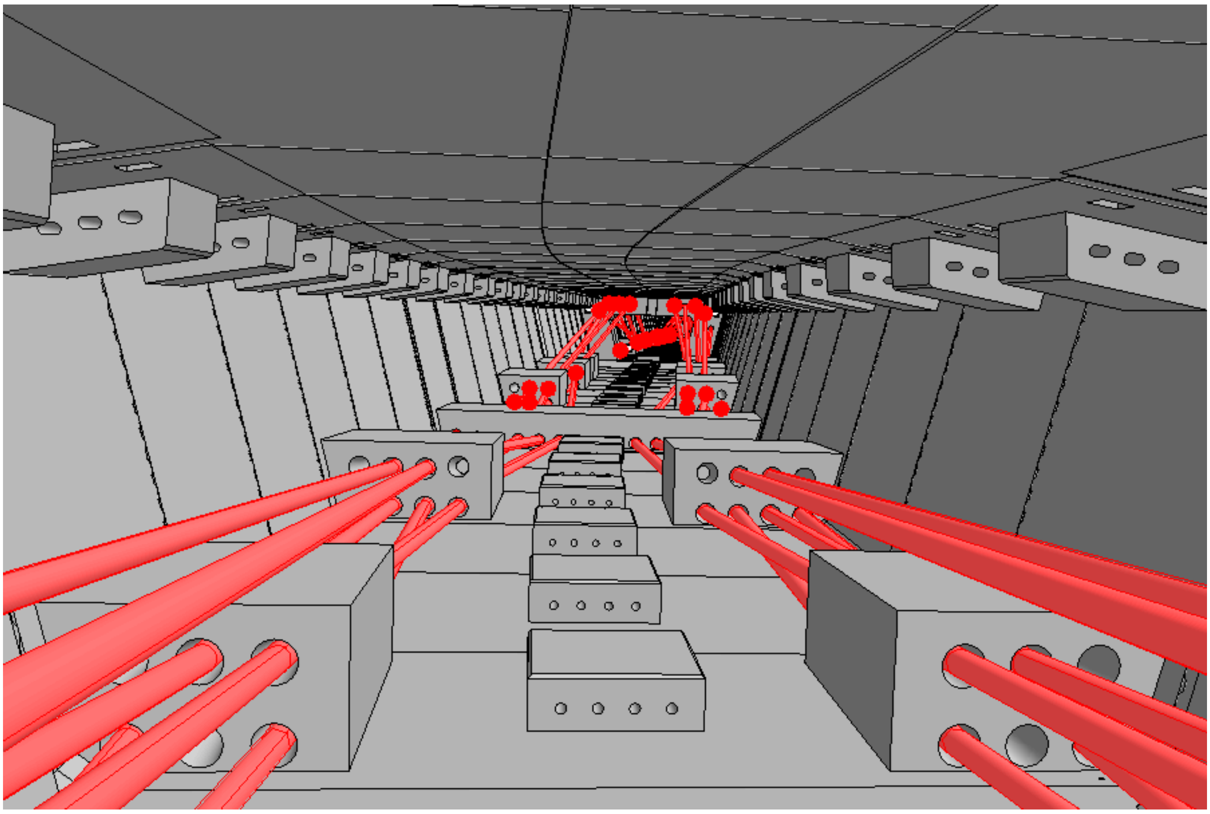
Precast Segment Reinforcement

A significant part of the design period was spent developing the reinforcement design of the bridge deck segments to facilitate a semi-automated method of assembly of the segment reinforcement cages. This required detailing of the transverse reinforcement bars in the segments so they could be assembled as “slices” of reinforcement bars, i.e. bars that formed the cross section of the bridge deck that could be tack welded together with a minimum number of side-by-side bars to ensure sufficient gaps between the slices to allow for easy concreting.

External Tendons

The prestressing layout of this balanced cantilever bridge is unusual in that all span continuity tendons are external to the concrete segment cross section. The tendons are anchored in the pier diaphragm walls and deviated to achieve the designed profile using concrete deviator blocks in the corners of the web and bottom flange junctions. A view of the layout of the tendons inside the box girder is shown in figure 10 below. This arrangement was designed in accordance with the preferences of MCD, who found that casting of the bridge segments would be simplified if no tendon ducts were cast into the bottom flange of the segments. Such bridges have however been constructed before, most notably the Ile de Re bridge in the Charente-Maritime region of France on the Western coast.

Figure 10 - 3D view of external tendons inside box girder



Balanced Cantilever Method / False Cantilevers

Balanced cantilever construction of the deck starts with erection of deck segments at the piers and proceeds outwards progressively towards the middle of the spans. This works well for a continuous bridge with many spans, but there is always a problem at the end of the continuous spans at the movement joints between the 5 or 7 span bridge modules. How are these end spans to be erected when there is a movement joint and a physical gap between the segments at the pier?

The answer is to temporarily connect the movement joint pier segments together using prestressing and to erect those segments in cantilever using the same construction technique as all the other spans. This "false" cantilever method avoids the use of falsework that would otherwise be required to support these end span segments; falsework which would have been very hard to accommodate with the temporary bridge underneath the box girders.

Control of Bridge Cantilever Deflections

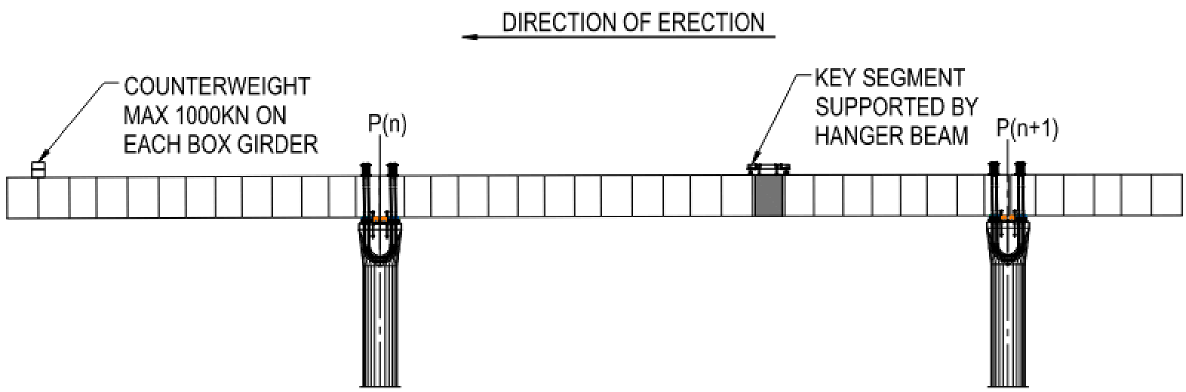
Although it is called balanced cantilever construction, during the erection of the segments the cantilever goes temporarily "out of balance". It is not possible to erect a pair of segments on each side of the pier simultaneously, so for a short period there is always one more segment erected on one side of the pier compared to the other. This out of balance causes a net bending moment on the pier and monopile, a moment that increases in magnitude as the cantilevers get longer towards the midspan and a moment that causes the top of the pier to deflect longitudinally towards the longer side.

The structural analysis showed that this deflection was potentially quite significant due to the tall height of the piers and the weak layers of the soil surrounding the top sections of the piles. Such large deflections would have adversely affected the lifting frames that were supported on top of the leading edges of the cantilevers – the lifting frames had been designed to be supported on both adjacent box girders and were used to erect segments for both box girders in a sequential fashion.

A method therefore had to be found to control the deflection to reasonable values. This was achieved by sharing the out of balance load of the segments between the adjacent box girders by constructing the deck slab between the box girders over the pier region prior to starting the cantilever erection with the lifting frames. The deck slab was effective in sharing the loading between the piers and thus any differential deflections between the adjacent cantilevers.

The deflections were further reduced using counterweights placed on the opposite side of the cantilevers to the out of balance, as shown in figure 11 below. Bundles of prestressing coils of strands were used as the counterweight, placed on the top surface of the cantilever, and a range of offsets from the pier centreline was calculated together with the corresponding correcting deflection that the offsets would create.

Figure 11 - Counterweight layout for control of bridge deflections



It is worth noting that when it comes to soil structure interaction calculations such as these, the observed deflections are in the author's experience always less than calculations using the engineering properties of the soil would predict. This was also the case for this bridge. It was for this reason a permissible range of offsets for the counterweight was provided so that MCD could move the counterweights as required to get a balancing deflection.

5. Conclusion

The design of the New Bridgewater Bridge was intricately linked to its planned construction method and required the designers to integrate the temporary works design and the construction method into the development of the structural design. Whilst an infrastructure project of this scale will always have challenges, the key to the project's success has been to identify those challenges and design them out wherever possible. Almost every aspect of this bridge was developed with a view to simplify the construction details and some notable successes are the construction of what is likely to be some of the longest large diameter piles in Australia with a neat solution for the pile / pier interface that took construction work above the water level and provided a safer working environment.

6. References

1. AS5100.5 Bridge Design: Part 5 Concrete, clause 10.4.2
2. AS5100.6. Bridge Design: Part 6 Steel and Composite Construction, clause 4.2.2.1
3. HK Code of Practice for the Structural Use of Steel, table 8.9

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