Design of the Main Spans, Second Gateway Bridge, Brisbane

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SYNOPSIS

The Second Gateway Bridge crosses the Brisbane River to the east of the city of Brisbane and is of overall length of 1627m with a main span of 260m and adjacent spans of 162m. The superstructure for the three main river spans comprises a twin cell prestressed concrete box girder constructed by the cast in-situ balanced cantilever method with segments varying from 15.57m to 5.2m deep and 3m to 5m long. The main spans are supported on large groups of 1.8m diameter bored piles and twin blade piers. The design of the Second Gateway Bridge was delivered by an integrated design team of Maunsell AECOM and Cardno for the Maunsell SMEC Joint Venture, with Coffey Geosciences performing foundation design.

This paper describes the design development process for the main river spans including the determination of the design criteria, selection of the superstructure cross section and structural form for the substructure. Details are provided of the investigations and testing carried out to confirm the design assumptions and how the design was prepared to facilitate the subsequent construction phase.

1.0 GATEWAY UPGRADE PROJECT

In September 2005, the Queensland Motorways Limited called for tenders for the upgrade of approximately 25km of the Gateway Motorway including a second bridge over the Brisbane River. The successful consortium was required to design, build and maintain the upgraded facility for 10 years. The upgrading required duplication of roadway and bridges south of the Brisbane River, a new bypass north of the Brisbane River, and a new second crossing of the Brisbane River to allow traffic to be converted to one-way northbound on the existing Gateway Bridge, and one-way southbound on the new, Second Gateway Bridge.

A consortium comprising Leighton Contractors Pty Limited and Abigroup Contractors Pty Limited formed the Leighton Abigroup Joint Venture (LAJV) to successfully bid the project which was awarded in October 2006. The design and construction of the Second Gateway Bridge is under the control of the Gateway Bridge Alliance comprising the Leighton Abigroup Joint Venture and VSL Australia. The design of the Second Gateway Bridge was delivered by an integrated design team of Maunsell AECOM and Cardno for the Maunsell SMEC Joint Venture, with Coffey Geosciences performing foundation design. The design commenced in October 2006, was completed during 2007 and the construction of the works is scheduled to be completed by August 2010, following which the existing Gateway Bridge will be refurbished to allow completion of the motorway upgrade.

2.0 DESIGN REQUIREMENTS AND DESIGN PHILOSOPHY

The Second Gateway Bridge is constructed just 50m to the east of the existing crossing and was required to have substantially the same appearance as the existing bridge, except it was also required to have:

- The same structural form.
- Provide for six lanes of southbound traffic and a 4.5m wide shared pedestrian/cycle path on its eastern edge.
- Piers of the same form but the width could match the width of the box girder.
- All piers in the same positions.
- A river navigational clearance that matches the existing bridge,
- A bridge height that sits below the obstacle limitation surface of the nearby Brisbane Airport, and
- A 300 year design life for durability.

These requirements were specified in the Project Scope and Technical Requirements (PSTR). This document implied the new bridge would be constructed using similar techniques to the existing bridge and was directed at achieving a continuous prestressed concrete bridge with no joints and continuity of reinforcement, thought to be the most durable bridge form and one more able to achieve the long life targeted by the 300 year design life. Although the existing bridge remains in excellent condition and is a credit to its designer and constructor, replicating this form was not considered appropriate for the Second Gateway Bridge because of the abundance of bearings and the resulting maintenance and replacement issues, and also because the structural form has a fundamental lack of redundancy. Advances in technology that reduce construction cost and time as well as increasing durability were sought. This approach was embraced by the client and some of the requirements in the PSTR were modified accordingly.

The design philosophy of the Second Gateway Bridge has sought to maximise service life and address the issue of the lack of redundancy of the existing bridge. Current technologies and materials are applied to achieve these goals and also reduce construction time and cost.

3.0 BRIDGE DESCRIPTION

3.1 Form and Articulation

The Second Gateway Bridge comprises 18 spans and is of overall length 1627m between abutments. The span configuration has been chosen to match that of the existing Gateway Bridge, with piers and abutments at the same alignments, except for the two piers that flank the main span. The structure is divided longitudinally into four modules of lengths 287m in the southern approaches, 701m for the main river crossing and two modules of lengths 352m and 287m in the northern approaches (refer to Figure 1). Each module comprises a continuous frame with the superstructure integral with the piers. The deck modules are separated by three superstructure halving joints incorporating deck expansion joints and guided sliding pot-type bearings. At both abutments, guided sliding pot bearings provide for longitudinal expansion. The halving joints provide a means of accommodating relative longitudinal displacements and have been positioned away from the piers, as in the existing bridge. This is structurally efficient, minimizes the number of bearings, and keeps the tops of the piers compact in width.

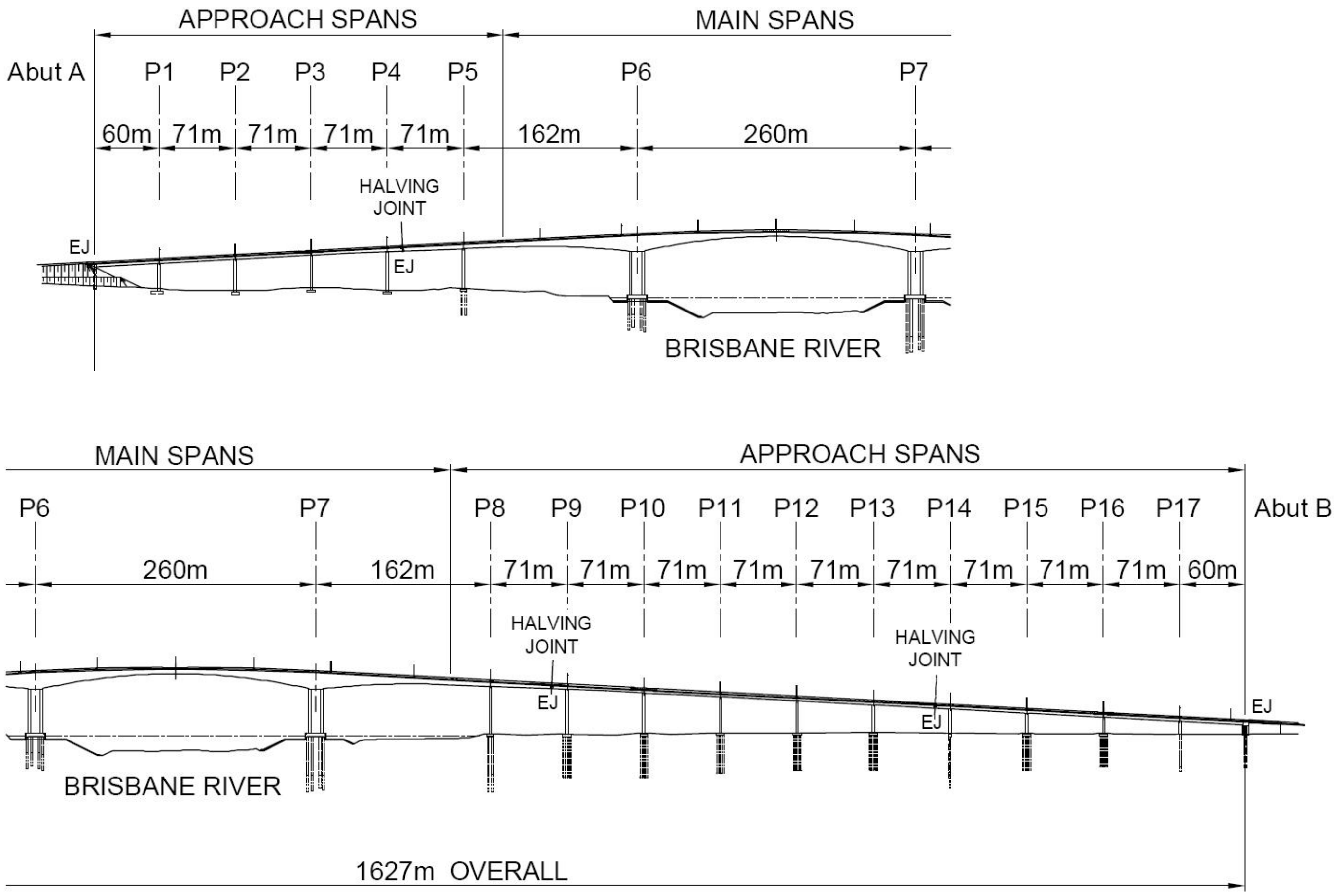

Figure 1 – Bridge Elevation

The vertical alignment of the main span superstructure is dictated by the road geometry and the geometric restrictions of the shipping navigational and aircraft clearance envelopes, and matches the existing bridge. The superstructure each side of the main span is on a constant grade of 5.3% with a vertical circular curve provided between the main river piers.

3.2 Superstructure

The main span bridge superstructure comprises a two cell prestressed concrete box girder with vertical webs, supporting six 3.5m wide traffic lanes, two 0.5m wide shoulders and a 4.25m wide shared path separated from the traffic lanes by an intermediate concrete traffic barrier. The overall deck width is 27.5m which is wider than the existing bridge because of the presence of the shared path. The deck has two-way 2% deck crossfall with the crown at the middle of the box (refer to Figure 2). The depth of the box girder varies from 15.57m at the two main river piers, to 5.20m deep at mid-span of the main span, and transitioning to a constant depth of 3.3m in the approach spans.

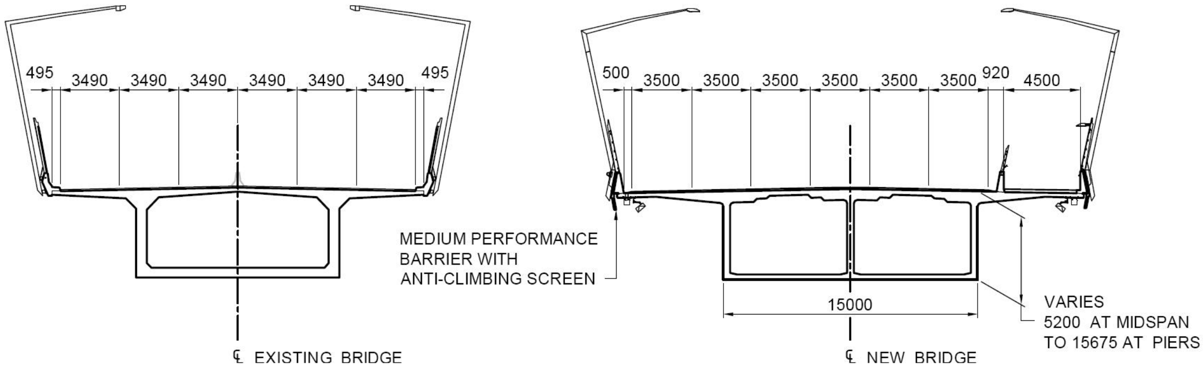

Figure 2 – Main Span Cross Section

At the two main piers, two-cell pier-boxes integrate the superstructure with the piers. Each pier-box comprises 2.5m thick transverse diaphragms which are extensions of the twin blade pier columns. In the longitudinal direction, each of the webs has been increased in thickness to 1.0m wide. This arrangement of webs and transverse diaphragms provides a direct transfer of girder shear forces into the substructure.

3.3 Substructure

Each river pier consists of twin, box section, reinforced concrete columns of outer dimensions 15.0m by 2.5m, supported by a reinforced concrete pilecap, which in turn is supported by 24 No. 1.8m diameter bored piles. The twin blade piers are similar in appearance to the existing bridge twin blades, and provide the necessary longitudinal flexibility in-service, whilst providing adequate stiffness and stability for the construction stage cantilevering (refer to Figure 3). The design of the twin blade piers was governed by in-service conditions, and the effects of second-order geometric non-linearity were considered.

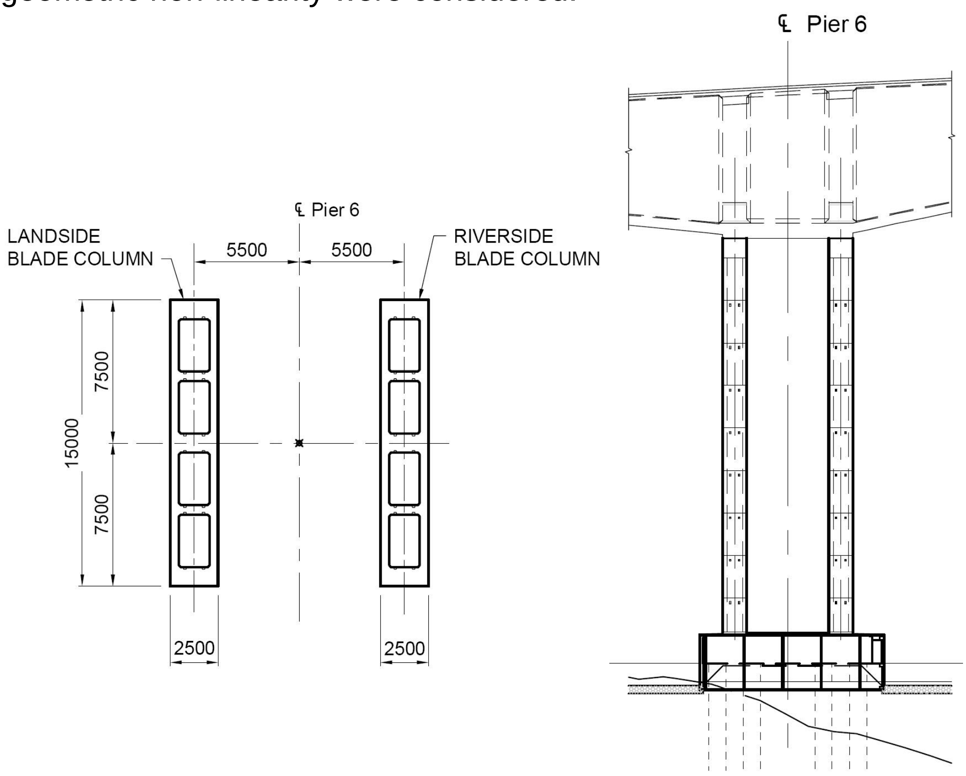

Figure 3 – River Piers

The main pier pile caps have a plan area of 19.5m by 17.6m and are 3.2m thick. The pile caps support an outer pier protection system (refer to Figure 4). The tops of the pile caps have been set at Reduced Level 4.2m, which allows construction of the pile caps to be carried out above water using land-based methods. Considerable savings in construction time resulted from this cost effective solution, which also eliminates the additional risks associated with marine-based work and construction below river water level. Precast skirt units attached to the pile caps for pier protection from low impacts also provide the necessary enclosure of the piles during low tidal levels.

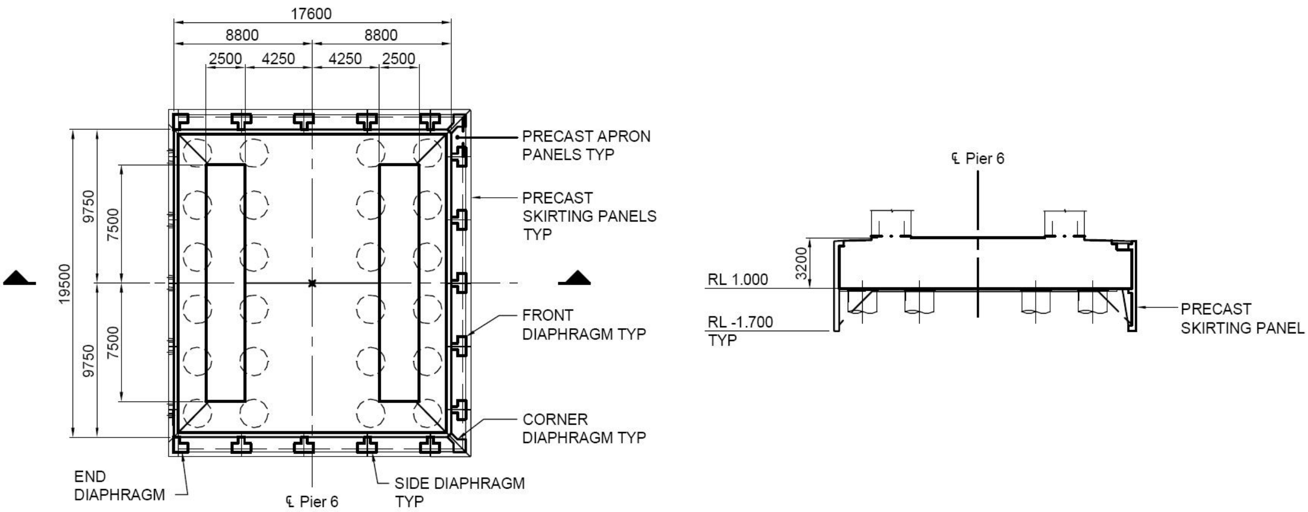

Figure 4 – River Pier Pilecaps

The pile layout comprises two groups of piles directly located under each pier blade at 3.3m centres transversely and 3.6m centres along the axis of the bridge. The piles are 1.8 m diameter bored piles, cased through the upper soft alluvial and clay layers. The casing achieved a seal in the upper zone of a carbonaceous siltstone layer and coring continued through that layer and into the inter-bedded siltstone and sandstone layers below, to form the rock sockets that found at depths ranging between 25-55m depth and provide vertical load capacity via a combination of side friction and end bearing.

4.0 DESIGN CRITERIA

4.1 General

The design criteria were based on the Australian Bridge Design Code AS5100 (Ref 1), supplemented by additional requirements nominated by the Main Roads Department, Queensland for Queensland Motorways. Reference was also made to the AASHTO Load Resistance Factor Design (LRFD) Code (Ref 2) for inclusion of additional loadings applicable to the construction stage for this form of bridge structure and to the design requirements for shear and torsion appropriate to large box girders.

4.2 Design Life

The PSTR specified that the Second Gateway Bridge primary structural elements shall have a design life of 300 years for durability. As established guidelines, and the guidance in AS 5100 in particular, address a design life of "only" 100 years, it was therefore necessary to take a first principles approach based on “building in” the required durability at the outset, where feasible, and minimising the need to take measures later in the life of the bridge to achieve 300 years of service. Integral with this philosophy was the appropriate selection of high quality materials chosen to address the particular durability issues that are posed by the range of exposure classifications experienced by the bridge elements. The selection of high quality concrete (generally 50MPa), cover of 55mm (generally) and the typical use of “black” steel reinforcing with the selective use of stainless steel in splash zone elements, was the general philosophy adopted. This approach is described in more detail in a companion paper.

4.3 Dead Loads

The dead load condition was determined by stepping through the proposed construction sequence for the main spans and cumulating stress resultants. The time dependent effects of creep and shrinkage were incorporated in the determination of the long-term dead load condition. The creep and shrinkage behaviour of the concrete for the superstructure was the subject of testing to ensure these properties were realistically predicted in the design. This testing was seen as a good safeguard against unexpected long term creep behaviour of the bridge. This testing was conducted on larger size test samples and led to a modification of the shrinkage strains that would otherwise be determined from the Australian design code, but the code values for creep strains were retained.

4.4 Live Loads

The bridge was designed for six lanes of the SM1600 traffic design load in accordance with AS5100. In addition, consideration was given to the HLP400 heavy load platform, which was restricted to travel between the outer box girder webs. The HLP400 loading was applied concurrently with half of four design lanes of SM1600 traffic loading.

The above design live loads were applied in combination with a global shared path loading of 3kPa. A 7kPa localised shared path live load was also applied on any 30m length of the footway.

Notwithstanding the fact that the shared path is located on the eastern side of the Second Gateway Bridge, each box girder cantilever was designed for the same traffic loading to give flexibility for possible future traffic use.

4.5 Balanced Cantilever Construction Loads

Under the assumed balanced cantilever construction of the superstructure, the following additional loadings were considered:

- Asymmetric concreting of one full segment on the river side of the main piers;
- Formwork travellers for the main spans, assumed as 265 tonnes;
- Construction live load of 0.25kPa;

- Density variation from one cantilever to the other of 2% to account for systematic errors in concrete dimensions;
- Differential wind loading from one cantilever to the other of 100% and 60% of the construction wind load;
- Allowance for 50mm vertical jacking to correct potential misalignment of the main span cantilever arms prior to casting closure segments;
- Loss of traveller case.

Load factors and load combinations for the above construction loads at the ultimate limit state were taken from the AASHTO LRFD design code.

4.6 Wind Loads

A 2000 year average return interval was adopted for the in-service ultimate limit state together with a 20 year average return interval for the in-service serviceability limit state. The average return interval for the construction ultimate limit state was adopted as 40 years. The corresponding regional basic wind design wind speeds were determined using AS5100.

Drag coefficients for the superstructure were determined from AS5100 and compared to the coefficients adopted in the design of the Existing Gateway Bridge These were found to correlate closely.

The effect of wind turbulence and buffeting due to the close proximity of the new bridge to the existing bridge was investigated via sectional models in a wind tunnel. No instability of either structure (depending on the direction of the wind) was found up to wind speeds beyond the ultimate limit state wind speed (63m/sec).

4.7 Earthquake Loads

A design acceleration response spectrum for the bridge was developed assuming an acceleration coefficient of 0.10 (100 year design life), an Importance Factor of 1.25 (structure essential to post-earthquake recovery) and a site factor specifically determined for the site. A three dimensional response spectrum analysis was carried out using a structural response factor of 2.8. The use of a low structural response factor allowed simplified reinforcement detailing to be used.

4.8 Ship Impact Protection

The first line of defence against ship impact on the main river piers is the submerged arrestor islands that are created by reducing the construction platforms from which the piers are built. The width of the arrestor islands provide sufficient river width for navigation and have a clearance to the face of the pier columns to prevent the bow of an Oriana Class passenger liner from piercing the pier columns. The length of the islands was determined from the recommended best practice indicated in the AASHTO Guide Specification for Vessel Collision Design (Ref 3) that indicates that the use of arrestor islands has been proven as the safest and most effective means of providing pier protection (refer to Figures 5 and 7). The extent of the arrestor island is different at each pier, in recognition of the different levels of risk presented by the different pier locations relative to the shipping channel

In addition, the pile caps and pile groups at the main river piers have been designed to resist an ultimate lateral impact force of 20,000kN applied either perpendicular or parallel to the longitudinal axis of the bridge. The pile caps are finished with precast panels to provide a clean finish and to ensure the pile cap and piles are not exposed at low water levels. These panels also provide resistance to small impacts from pleasure vessels and provide some energy absorption characteristics by way of crushing, should the pier experience an impact from a medium sized vessel of low draft that may be able to pass over the top of the arrestor island at high tide.

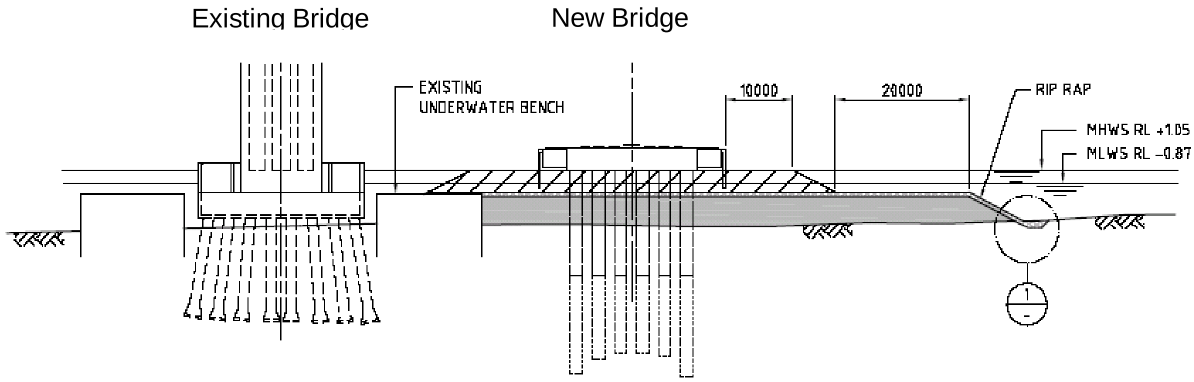

Figure 5 – Cross Section through Pier 6 Ship Arrestor Island

5.0 DEVELOPMENT OF CONCEPTS

Within the constraints of the PSTR, a number of options were considered in the development of the final tender design for the main spans. These included the following:

- Variation in lengths for the spans adjacent to the main span. The Existing Gateway Bridge has span lengths of 145m, 260m and 145m for the three main spans, with 88m transition spans between the three main spans and the regular 71m approach spans. The extension of the 71m span lengths (with corresponding increase in side spans to 162m) best suited the construction methods proposed;
- Use of a single cell box girder. Consideration was given to retaining the same width of bottom flange as the Existing Gateway Bridge (12m) and in adopting a wider 15m bottom flange. For the overall width of the new bridge required, it was determined that a two cell box girder was more economic.
- Vertical and inclined webs. Preliminary designs indicated that the use of a two cell box girder with either inclined or vertical webs resulted in similar construction costs. It was considered that vertical webs provided a structure of closer appearance to the Existing Gateway Bridge and simplified the design of the form travellers.
- Pile size. Preliminary designs for pile groups comprising 1.5m, 1.8m, 2.0m and 2.5m diameters were carried out. A linked pile caisson option was also considered using similar technology to offshore oil platform construction. The 1.8m diameter solution was determined to be the most cost effective solution.

6.0 MAIN PIER FOUNDATIONS

6.1 Pile Design

Stress resultants to the main river pilegroups under in-service and construction loadings were determined using 3D global analysis models of the bridge structure. A CLAP analysis (derivative of DEFPIG) was carried out to consider pile group effects and determine design axial loads and bending moments in the individual piles. Pile axial capacity was assessed using the method of Rowe and Armitage, as required by the PSTR, which is specifically intended for socketed piles in relatively weak rock. Conservative design parameters were adopted including the use of a geotechnical strength reduction factor of 0.45, a further reduction factor for end bearing of 0.5 and limiting the vertical displacement to 25mm. Maximum ultimate design axial load S* on the 1.8m diameter piles was 35.2MN and all piles were designed with a factor of safety on working loads in excess of 3.0. During construction, bore logs were taken at each individual pile, two large scale test piles were used and a load test of 1.2S* was applied to one pile in each pile group to confirm the pile axial capacity.

6.2 Pilecap Design

The two main river pilecaps were designed using strut and tie analysis methods. The multiple layers of bottom reinforcement were detailed using bundles of three bars to minimise potential congestion from the vertical pile reinforcement, which was also detailed using a combination of 75mm diameter stressbars and conventional reinforcement (refer to Figure 6).

To alleviate adverse thermal differentials and high peak temperatures during the casting and hydration of the large pile caps, a system of cooling pipes was installed. Ambient air was pumped through these pipes at relatively high velocity to extract heat from the pilecap core. The pile cap was instrumented to show that this system effectively maintained temperature differentials to 25°C, and peak temperatures of 81°C were reached at isolated points in the concrete mass.

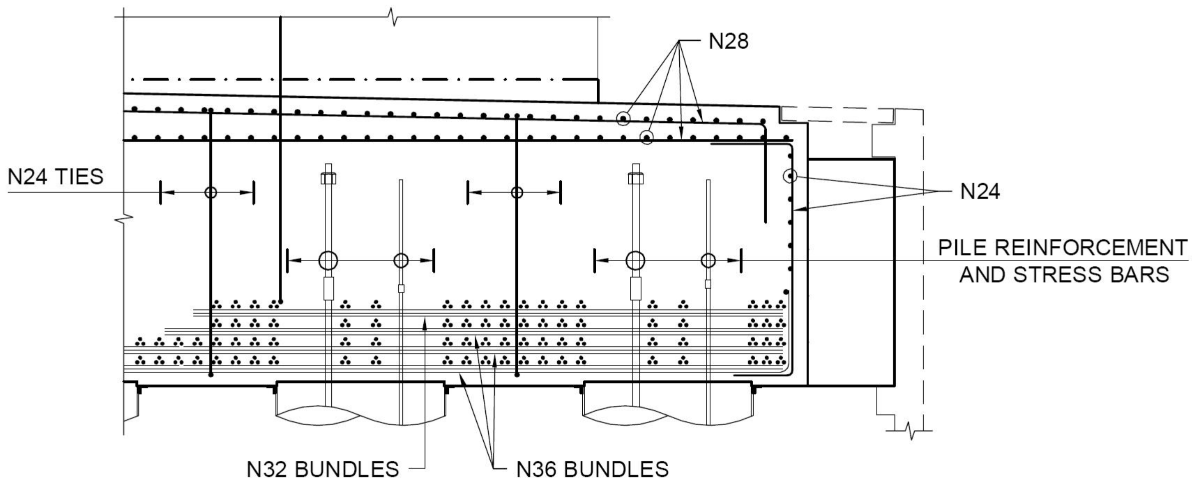

Figure 6 – Part Cross Section Through River Pier Pilecaps

Figure 7 – View of Pier 6 Showing Construction Platform (Future Ship Arrestor), Pile Cap and Column with Pier Head Complete, Ready for Balanced Cantilevering

7.0 SUPERSTRUCTURE

The twin cell box girder has a 15m wide constant-width bottom flange. Bottom flange thickness in the main spans varies parabolically from 1040mm at the main pier faces to 300mm at mid-span of the main span and 375mm at the side span closure segments. The thickness of the three webs steps from 500mm wide at the main piers to 400mm at the ends of the cantilevers in steps of 50mm. A stability check was carried out on the tall thin webs.

At the two main piers, two-cell pier-boxes integrate the superstructure with the piers. Each pier-box comprises 2.5m thick transverse diaphragms which are extensions of the twin blade pier columns. In the longitudinal direction, each of the webs has been increased in thickness to 1.0m wide. This arrangement of webs and transverse diaphragms provides a direct transfer of girder shear forces into the substructure. Access openings 2.1m high by 2m wide are provided through the transverse diaphragms in each box girder cell.

A partial prestress design was used for the main span superstructure. The in-service stress limit cases were:

- Maximum concrete compressive stress of $0.4f'_c$ under permanent effects and $0.5f'_c$ under peak serviceability limit state load cases;

• Zero tension under permanent loads and half live load (as per AS5100 for B2 Exposure Classification)

The typical distribution of permanent stresses in the superstructure are indicated in Figure 8. for the long term case following all losses and creep redistribution.

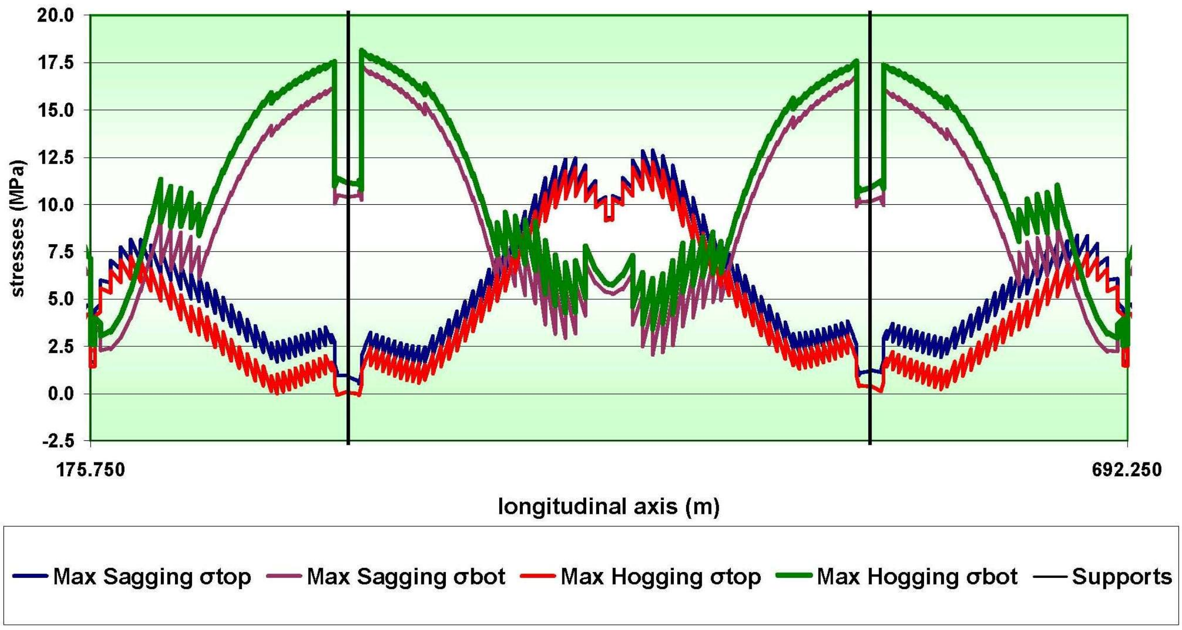

Figure 8 – Permanent Dead Load Stresses in the Superstructure

The bottom flange thicknesses were determined from consideration of serviceability stress limits. The web thicknesses were determined from consideration of combined shear, torsion and transverse bending effects. The top slab dimensions were dictated by the requirement to accommodate the large number of top cantilever prestressing tendons and to achieve the required transverse bending strength of the outer cantilevers.

Segment lengths were initially set at 3.0m long for the first 10 segments out from the main piers, then 10 segments of length 4.0m and 10 segments of length 5.0m. This segmentation was chosen to limit the wet concrete weight being carried by the form travellers and the out-of-balance moment carried by the piers. At the end of the land side cantilevers, the twin cell box girder transitions to the twin box girders of the approach spans within the closure segment, which incorporates a transverse diaphragm. To achieve a balanced condition at the end of cantilever construction of the approach spans about the transition pier, the last 5m cantilevered segment in the main spans on the land side is reduced in length to 2.75m. The resulting imbalance of the main cantilevers was restored by increasing the web and bottom slab thicknesses of the land cantilever over the last ten segments.

Longitudinal prestressing tendons in the top slab each comprise 17 and 19/15.2mm diameter extra high tensile strands. Typically at each segment end, six tendons are anchored in face anchors adjacent to the web. The tendons are located in three layers, symmetrically positioned about each web. There are 60 cable ducts per web with two as spares. Bottom flange longitudinal tendons each comprise 19/15.2mm diameter extra high tensile strands, which are anchored in internal blisters located adjacent to the webs. The plan layouts of the tendons were detailed to accommodate the penetrations in top and bottom flanges which are required to support the form traveller. Transversely, the top slab is post-tensioned using 5/15.2mm diameter extra high tensile strands spaced at 1.0m centres along the main spans. The transverse prestress is provided to balance permanent loads and control cracking under traffic loading.

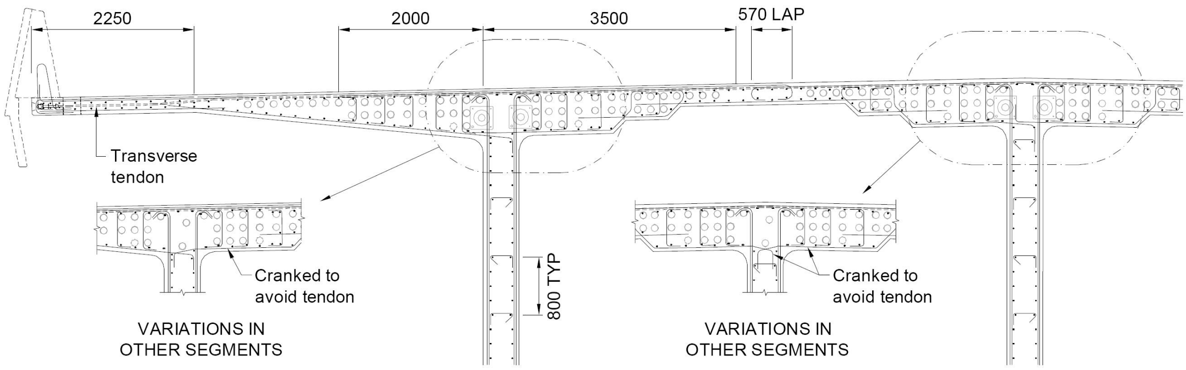

Figure 9 – Typical Box Girder Top Flange Details

Vertical post-tensioning of the webs has been restricted to areas of the webs adjacent to the bottom flange prestressing anchorages, to control potential cracking in the webs. At these locations, 32mm diameter stressbars on the web centreline at 600mm centres have been adopted.

The pier boxes are post-tensioned longitudinally at regular intervals down the webs, vertically in the webs and diaphragms and transversely in the bottom slab.

The design for shear and torsion involved recourse to the AASHTO LRFD code because the Australian code does not deal well with this issue. The AASHTO LRFD Ed 3 rules for combined shear and torsion at the ultimate limit state were used with a resistance factor of φ =0.7. A review of a number of international codes was undertaken to confirm this was an appropriate design approach. In order to ensure satisfactory performance at service loads a principal tensile stress limit of $0.289\sqrt{f'_c}$ at a serviceability load case of permanent loads plus half live load was adopted.

The transverse reinforcement was detailed as slices at 200mm centres to facilitate prefabrication of reinforcement cages. The longitudinal reinforcement in the top flange was also designed to allow launching of the form traveller prior to stressing the cantilever tendons for the newly cast segment.

8.0 SUMMARY

The Second Gateway Bridge is a prestressed concrete bridge with the three main river spans of the same structural form and of similar appearance to the Existing Gateway Bridge. The design philosophy has been to closely match the geometry of the existing bridge to achieve a similar visual appearance, to satisfy the design requirements of the PSTR, to incorporate current world's best practice in the technical design and methods of construction and to provide a robust, durable structure appropriate to the bridge's significance in the Brisbane landscape.

ACKNOWLEDGEMENT

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