Design of the Approach Spans to 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 approach spans with span lengths of 71m. The superstructure for the approach spans comprises a pair of prestressed spine girders of matchcast segmental construction and erected by the balanced cantilever method with segments typically 2.8m long and varying between 3.3 and 5.2m deep.

This paper describes the design development process for the approach 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. GATEWAY UPGRADE PROJECT

In September 2005, the Queensland Motorways Limited (QML) 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 included a new 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 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 geotechnical 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. DESIGN REQUIREMENTS AND CONCEPT DESIGN

The Second Gateway Bridge is constructed 50m to the east of the existing crossing. The Project Scope and Technical Requirements (PSTR) at tender included the following requirements:
- Provision for six lanes of southbound traffic and a 4.5m wide shared pedestrian/cycle path on its eastern edge with several widenings for rest spots.
- Similar appearance and structural form to the existing bridge.
- Match cast segmental construction not permitted.
- 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.

The PSTR pointed towards a prestressed concrete box girder with reinforcement continuous through the segment joints, like the existing bridge.

Although the existing bridge remains in excellent condition, 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.

LAJV favoured erection of the approaches by the balanced cantilever method, and also match cast construction. These construction preferences, together with durability and other considerations led to the following major changes to the PSTR being proposed and eventually embraced by QML:
- Piers were made monolithic with the foundation and the superstructure, whereas the existing bridge piers are pinned top and bottom. This facilitated balanced cantilever construction and avoided a difficult maintenance task in eventual replacement of pier bearings.
- Movement joints were located at halving joints at the contraflexure points in spans 5, 9, and 14 and also at the abutments, with multi-span continuous portal frame structures between the joints. This articulation is markedly different from the existing bridge.
- Match cast construction was adopted with two separate precast segments in the cross section connected by a stitch pour. LAJV were able to use segment moulds from a previous project. Durability concerns were addressed by a requirement for minimum compression at the joints of 1 MPa for serviceability load cases, an epoxy filled groove in the deck at the joints, and an extra layer of waterproofing.
- The main side spans of the bridge were increased from 145m to 162m so that the adjacent approach spans could be the standard 71m span rather than the special 88m span of the existing bridge.
- One span was eliminated at the northern end of the bridge by using a piled embankment at an adjacent zone of high potential settlement.

The form of the new bridge is considered to be robust, durable and suitable for efficient construction. The form of the cross section and the continuous frames and halving joints did introduce some interesting aspects to the design.
3. 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 side spans. The structure is divided longitudinally into four modules of lengths 287m in the southern approaches, 698m for the main river crossing and two modules of lengths 355m 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 at approximately 1/5th of the span away from the piers. This is structurally efficient, minimises the number of bearings, and keeps the tops of the piers compact in width.

The major benefit of this articulation is the omission of bearings at the top and bottom of each pier which was adopted in the existing Gateway Bridge. It is considered very difficult to replace bearings located at the base of the piers and, given the 300 year design life, a philosophy of minimising the use of bearings was adopted to minimise future maintenance. The solution has the advantage of eliminating the high cost of on-going inspection and maintenance of bearings located at the tops of piers, and the inspection and maintenance of bearings and expansion joints at the halving joints can be achieved from access provided within the box sections. The resulting bridge articulation is also a more robust solution whereby, unlike the existing Gateway Bridge, the new bridge is immune from progressive collapse should one pier fail.

Figure 1a – Bridge Elevation – Southern Approach + Main Spans
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 deck 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 superstructure of the approach spans comprises twin spine beams of single cell box girder of 3.3m depth, tapering to 5.2m depth over the spans adjacent to the main spans. To match the box section of the main spans, the overall width between the outer edges of the twin box spines is 15m, and the approach pier width of 9.2m is identical to the existing bridge which ensures a similar appearance as well as avoiding excessive pier stiffness. However, the resulting geometry results in severe indirect support at the piers.

The two box sections that form the approach spans are mirror images of each other and are highly asymmetric sections. The segments are approximately 2.8m long, constructed using a “short line” match casting method in a set of casting cells from a casting yard established on-site. The section was chosen based on the past successful experience of the design and construction team on the Westlink M7 project in Sydney. The twin box sections were separately cast and erected, resulting in smaller and lighter sections to be transported and erected, saving costs in erection equipment and also in ground improvement of the construction platform below the bridge. A total of 742 segments were produced for the approach spans.

The segments have a 5400mm bottom flange width with outer edges aligned with the main spans. The box sections are cast vertically with the webs vertical and top and bottom flanges horizontal. The segments are then subsequently rotated to a 2% crossfall to achieve the crown of the carriageway, and stitched together with an in-situ longitudinal pour. The cross section details were optimized to minimise the number of casting cells which would be required to cast all segment variations, including the variable depth segments (refer to Figure 2). Of particular note is the extreme asymmetry of the box sections. This resulted from the need to align the
The typical segment depth is 3.3m with a varying depth up to 5.2m for the segments that connect to the main spans. This is deeper than the 3m section adopted for the existing bridge because the design traffic live load is now greater than the design loading used some 25 years ago when the existing bridge was constructed, and furthermore the PSTR required a 1MPa residual compression between segment joints under all serviceability limit state load cases.

The top flange thickness was governed by the requirement to locate the top flange prestressing anchorages within the flange depth adjacent to the webs. Between webs the top flange varies in depth from 375mm midway between webs to a maximum of 450mm adjacent to the webs. The bottom flange thickness varies from 350mm at midspan to 550mm adjacent to piers. The web thickness varies with the inner web being either 350 or 450mm wide, and the outer web having three thicknesses; either 450, 550 or 650mm. A typical segment cross section is shown in Figure 3.

Figure 2 – Approach Span Cross Section

Figure 3 – Approach Span Typical Segment Cross Section
The approach spans were erected using the balanced cantilever method and prestressed using tendons internal to the concrete section. The joints between match cast segments in the completed structure are coated with epoxy. Longitudinal deck prestress tendons comprise either 12 No. or 19 No. 15.2mm diameter extra high tensile strength strands.

Pier segments were cast in-situ while the remaining segments were erected by either crane or from a purpose made erection truss and temporarily connected to the previously erected segments using VSL stress bar. A 150mm long unreinforced concrete stitch is cast between the pier segment and the first precast ‘P+1’ segments. The remaining segments are match cast and erected with epoxy coated joints. When corresponding segments have been erected on each end of the cantilever, permanent prestressing tendons are installed within the box top flange and tensioned from one end. Once the cantilevers are completed, a short (200mm) in situ unreinforced stitch is cast between the two deck cantilevers followed by the installation of continuity tendons stressed from blisters formed in the top and bottom flanges. All permanent prestress is internal to the section and fully grouted. All stressing blisters for the continuity stressing and for the temporary erection bars are cast as part of the concrete box section. Top cantilever tendons and top continuity tendons are either 12 or 19/15.2mm strands. The bottom continuity tendons are typically 19/15.2mm strands. The first two segments from the piers have vertical PT to limit principal tensile stresses and control cracking and all segments have transverse PT in the top flange (typically three 5/15.2mm strands per tendon).

The four segments of each cantilever erection cycle are erected to ensure not more than one segment is out of balance either longitudinally or transversely. For some of the taller piers, a prop was required to prevent buckling of the pier during the erection stage and prior to mid-span continuity being achieved. In these cases the top of the pier was propped back to the previously completed structure.

3.3 Substructure

3.3.1 Piers

Piers comprise reinforced concrete twin cell voided columns 2.3m thick and of constant 9.2m width which are similar in size to the existing bridge. (Refer to Figure 4). At piers 1 to 4 the pier columns are supported on spread footings founded on rock close to the ground surface. Piers 5 and 8 are supported on pile caps of plan area 13.5m by 6.5m. The pile caps are supported by 10 No. 1.5m diameter steel lined bored piles. For piers 9 to 17, the columns are supported on large (40 to 45 number) groups of driven 550mm octagonal prestressed concrete piles, except at pier 14 and 17 where the pier columns are integral with a single line of 3 No. 1.8m diameter bored piles. The single line support at these locations provides a structure with greater flexibility to deal with longitudinal movements due to creep, shrinkage and elastic shortening without the piers attracting excessive longitudinal flexure.
3.3.2 Abutments

Abutment A is of the spill-through type with a batter height of more than 20m. It is founded on pad footings stepped at three levels on the transverse sloping rock surface. Six 1.5m diameter columns support the headstock, four of them directly under the pot bearings.

At Abutment B the bridge support and embankment retaining tasks are achieved by independent structures. The headstock is supported on four 1.5m diameter columns which are extensions of bored piles. The outer two columns are directly under the sliding pot bearings which carry the highest bearing loads. The inner two columns are offset longitudinally from the line of the bearings to increase the abutment stiffness. The columns are concealed by precast panels. The embankment is piled against settlement and retained by a reinforced soil structure. The 6m long relieving slab spans the narrow gap between the abutment and the retaining wall. The abutment is restrained longitudinally by anchoring the end of the relieving slab into the embankment with short bored piles.

4. ANALYSIS AND DESIGN

4.1 Design Criteria

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. One such requirement was that for match cast segmental construction, under the worst serviceability loadings,
there shall be a minimum compression at segment joints of 1MPa anywhere in the cross section. This requirement was imposed to provide a better guarantee that there would be no cracking in the cross section and that there was no tendency for epoxy joints to experience tension or open up under any service load condition.

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. Since established guidelines, and the guidance in AS 5100 in particular, address a design life of “only” 100 years, it was 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 conditions experienced by the bridge elements. The selection of high quality concrete (generally 40 or 50MPa) and appropriate concrete cover was the general philosophy adopted. This approach is described in more detail in a companion paper.

4.3 Design Loads

4.3.1 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 New Gateway Bridge, each box girder cantilever was designed for the same traffic loading to give flexibility for possible future traffic use.

4.3.2 Balanced Cantilever Construction Loads

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

- Maximum of one segment out of balance including a dynamic allowance for sudden segment release.
- Construction live load of 0.25kPa and a 60kN allowance for a stressing platform at the end of each cantilever.
- 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 20mm vertical jacking to correct potential misalignment of the cantilever arms prior to casting closure pours;

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

### 4.3.3 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.

### 4.3.4 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.

### 4.4 Substructure Design

A global analysis of the deck and piers was performed using in house software “iCreap”. The software performs a step-by-step analysis of construction staging allowing for time dependent effects such as concrete creep and shrinkage and prestress losses. The relieving effects of cracking in the piers was also allowed for in the analysis.

Separate 3-dimensional computer frame models were used to analyse the stability of each pier during construction. Piers 2 to 5, 8 to 12, 14 and 17 required propping during deck cantilevering to ensure stability during construction. This was achieved via a purpose made prop between the pier segment of the cantilever pier and the tip of the previously built deck cantilever. The prop also served as a construction access walkway between adjacent cantilevers.

Foundation stiffness was represented by linear springs in the global model. The spring stiffnesses were determined by Coffey Geoscience using “CLAP” software purposely developed for this project. The software performs a 3 dimensional analysis of pile groups allowing for material non-linearity. The development of appropriate linear springs for use in the global structure model was an iterative process since the spring stiffness depends on the level of loading in the foundations.
Foundation loads determined from the global analysis were then input into the pile group analysis using “CLAP” to establish loads in individual piles.

4.5 Superstructure Design

4.5.1 Deck

The severe asymmetry of the box sections and their erection as single spines before the longitudinal in-situ connection pour is completed, means that the principal axes of the segments are rotated by between 6 and 14 degrees depending on segment depth. Once the longitudinal median stitch between the two spines is cast the axis of bending changes as movement of each spine is constrained by the stitch. This resulted in non uniform longitudinal stresses across the top and bottom flanges of the box spines, increased shear forces in the outer webs and twist and horizontal deflections of the box spines during deck cantilevering. This effect complicated the segment design, necessitated thicker external webs to resist the higher shear forces that are attracted to them and resulted in uplift restraints being required for the inner bearings at the halving joints. It also meant that the centre of gravity of the cross section is not at the "centreline" of the box section and the segment self weight forces are not symmetrical, complicating segment lifting.

Due to the highly asymmetric geometry of the individual box spines particular attention was paid during the deck design to the behaviour of the boxes both during deck cantilevering and once construction was completed. Extensive use was made of 3 dimensional finite element analysis using either Strand 7 or ACES software to gain an understanding of the behaviour and to predict stresses and deformations in the boxes. Figure 5 shows the design longitudinal stress envelopes at critical points in the cross section for a typical approach span.

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. The code values for creep were however retained for the design of the structures.

The design for shear and torsion involved recourse to the AASHTO LRFD code because the Australian code was not considered appropriate for the design of large box sections. For the Second Gateway Bridge AASHTO LRFD Ed 2 (2002) [Ref 2] was adopted for shear and torsion design. However, in recognition of the prevailing Australian code requirements, the AASHTO provisions were modified. The resistance factors, \( \varphi \), in the AASHTO LRFD code are not the same as those in the Australian codes. The Australian codes traditionally recognise the more variable nature of shear and torsion and assign \( \varphi = 0.7 \) to the calculation of the shear and torsion strengths. These compare to the AASHTO LRFD \( \varphi \) values of 0.9 for shear alone, or 1.0 for shear in combination with torsion. Accordingly a resistance factor of 0.7 was adopted for the Gateway Bridge. In order to ensure satisfactory performance at service loads a principal tensile stress check was also used for the Second Gateway Bridge. This check adopted a principal tensile stress limit of
0.289\sqrt{f_c} based on a serviceability load case of permanent loads plus half live load. This criterion resulted in the requirement for some vertical post tensioning in the first two segments from piers in the approach spans.

4.5.2 Pier Segment

The cast in situ pier head segments provide a direct connection between the deck and piers allowing the structure to behave as a continuous frame. During construction the pier head also provides a rigid connection between deck and pier transmitting out of balance construction forces directly to the piers. The outer web of the deck is almost 3m beyond the edge of the pier resulting in indirect support of the heavily loaded outer web. Consequently, significant forces are required to be transmitted through the pier head segment.

The pier heads are transversely post-tensioned using 4 large tendons each comprising 37/15.2mm very high strength strands. (Refer to Figure 6). The tendons are stressed in a single stage from one end of the diaphragm to simplify construction. Additional post tensioning is provided at rest spot pier locations where the deck cantilever is thickened to accommodate the extra width of deck at these locations.

The location of the access openings through the pier diaphragm was critical to the performance of this element. The PSTR required access openings of minimum dimensions 900mm wide by 1600mm high to provide access for maintenance and inspection staff. The openings were located adjacent to the inner web as this

![Figure 5 – Longitudinal Stress Envelopes for a Typical Span](image)
ensured a relatively uninterrupted load path for the forces from the girder outer webs to the piers.

The pier head was designed using a combination of 3-dimensional strut-tie modelling and 3-dimensional finite element modelling using Strand 7. The finite element models were not only used to confirm the validity of the load paths assumed in the strut-tie models but were also used to check serviceability limit state behaviour. An example of a finite element model used in the analysis is shown in Figure 7.

4.5.3 Halving Joints

Three halving joints located in spans 5, 9 and 14 allow deck expansion and contraction to occur without overstressing the structure. The joints in spans 5 and 9 are located adjacent to the main river spans where the varying depth superstructure is approximately 4.0m deep. The deck thickness at the span 14 halving joint is 3.3m. Each halving joint comprises an upper and lower precast segment for each cantilever spine. The halving joint segments are the 4th and 5th segments from the pier, ie the
'P+4' and 'P+5' segments and are match cast against the ‘P+3’ and ‘P+4’ segments respectively.

During construction the halving joints are required to be “locked” up to permit cantilevering of the remaining 6 segments up to the midspan stitch. This is achieved through the use of temporary grout packers between the segments and 11 No. 75mm diameter stressbars between the upper and lower segments. Six of these stress bars are located internal to the box section and the remaining 5 between temporary concrete anchor blocks cast above the webs of the ‘P+3’ segments and upper halving joint segments (refer to Figure 8). The installation, stressing and de-stressing of the temporary stressbars is staged to ensure that stresses in the joint are kept to acceptable limits for all construction stages.

The joint segments were designed using strut-tie modelling with different strut-tie models used for the in-service and construction conditions due to the very different behaviour in these two states. The validity of the strut-tie models was checked using 3 dimensional finite element modelling to confirm the assumed load paths and also provide a means of checking serviceability limit state behaviour.

Additional serviceability limit state checks were undertaken based on the recommendations in BA 39/93 (Ref 3). This resulted in the need for additional inclined prestress in the joint to control cracking.

Figure 8 – Halving Joints
5. SUMMARY

The design of Second Gateway Bridge approach spans offered a number of interesting challenges to the design team. The construction method adopted resulted in an unusual structural form with highly asymmetric box sections during deck erection which complicated the design of the bridge. The design of the segment box girder bridges for this project also required development of additional design criteria for shear and torsion design aspects not fully covered in current Australian codes.

The bridge has been designed and detailed to achieve a long service life and is being built with particular emphasis on achieving the required cover, good concrete compaction and sufficient curing to realize the 300 year design life target.

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