Design of Lawrence Hargrave Drive Bridges

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Darrell Meyers is a Principal Engineer with Maunsell Australia and has over 17 years experience in bridge design. His technical experience spans the majority of bridge construction forms including precast segmental, balanced cantilever, incrementally launched, precast girders, cast-insitu, steel boxes, trusses and cable stayed structures.

Darrell was the lead bridge designer for the superstructure design of the balanced cantilever bridge of the Lawrence Hargrave Drive Project. Darrell was also responsible for the construction engineering and monitoring of balanced cantilever and incremental launched bridges during the construction phase of the project.

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SYNOPSIS

Over the last century Lawrence Hargrave Drive, between the coastal townships of Clifton and Coalcliff in New South Wales, has been plagued by rock falls and landslips. After a thorough investigation of options the final solution developed by the Alliance Team (RTA, Barclay Mowlem, Maunsell and Coffey) was to move the road away from the cliff face. This solution involved the design and construction of two back to back curved bridges, rockface stabilisation, retaining walls and road widening. The northern bridge consists of a 7 span 203m long prestressed double tee incrementally launched bridge. The southern bridge consists of a 5 span 448m long cast in-situ balanced cantilever bridge. The bridges carry two traffic lanes and a walkway up to 45m off the cliff face and up to 40m above the sea level. The project was completed in 24 months and opened to traffic three months ahead of schedule. The solution that was developed and refined by the Alliance team has demonstrated how the Alliance process can be utilised to obtain an optimum cost effective engineering solution. This paper highlights the design features of the curved incrementally launched and balanced cantilever bridges.

Figure 1 – Lawrence Hargrave Drive Bridges
1. BALANCED CANTILEVER BRIDGE

1.1 General

The 448m long balanced cantilever bridge consists of three 108m internal spans and 62m back spans. At the southern end of the bridge, the superstructure is supported by a spread footing abutment perched on the cliff face, at the northern end the superstructure is supported by the common pier with the incrementally launched bridge.

The superstructure consists of a single cell 6m wide box girder supporting a 13m wide deck slab. The depth of the box girder varies parabolically from 6m at the pier to 2.5m at midspan. The hollow 6.0m x 2.8m piers are up to 26m tall and are supported on six 1500mm diameter bored piles. The superstructure is integral with the piers and supported by twin sliding pot bearings at the abutments and the common pier.

Figure 2 – General Arrangement of Balanced Cantilever Bridge
1.2 Geometry Considerations

The horizontal geometry for the bridge was determined from a combination of accessible pier locations on the rock platforms, road geometry requirements and rockfall clearances. The resulting horizontal geometry from south to north consists of a 500m radius right hand curve reducing to a 350m radius followed by a transition into the reverse 150m radius curve of the incrementally launched bridge.

Vertically the bridge is on a constant 2.5% longitudinal grade. The super elevation is a constant 3% through the curved spans and transitions in span 5 to the match the reverse crossfall of the incrementally launched bridge.

1.3 Design Criteria

The design criteria for the balanced cantilever bridge is framed on the provision of the Austroads Bridge Design Code with the following modifications

- Design Load – T44 no HLP
- Shear and Torsion design – AASHTO LRFD Bridge Design and Construction Specifications 2004
- Allowable tensile stresses in the piers during cantilevering – Eurocode
- Load factors during cantilevering – AASHTO LRFD Bridge Design and Construction Specifications 2004

2. DESIGN FEATURES

2.1 Foundations

The foundations for the piers consists of six 1500mm diameter 15m long bored piles. The piles are socketed into class 3 sandstone and class 2 shales beneath the coal seam and required casing for construction. The piles support a 8.5x7.5x2.5m reinforced concrete pilecap. The reinforcement in the piles, pilecaps and piers are connected to the cathodic prevention system.

Providing access for the construction of the foundations was a major construction achievement, requiring the construction of an access road down the cliff face and along the shore line to the pier locations. During high seas, waves would break over the access road preventing construction.

2.2 Piers

The hollow rectangular piers measure 6.0m wide x 2.8m deep with a constant wall thickness of 0.5m. The piers vary in height from 21m to 26m. The wall thickness adopted took into consideration, cover requirements, reinforcement congestion, and concrete placement and compaction methods.

The 6m width of the pier was adopted to match the width of the box girder. The 2.8m depth of the pier was a compromise between the conflicting stiffness requirements for the cantilever construction method and the flexibility required for the completed in service design.
For the cantilever construction, the design criteria was to limit the tensile stresses in the pier concrete below two thirds of the flexural tensile strength of the concrete. This would prevent cracking of the pier concrete during construction to improve durability and provide better control over pier and cantilever deflections. To limit the tensile stresses below the design criteria limit a 3.2m deep pier section was required. However this would have resulted in a pier section that was too stiff to accommodate long term creep and shrinkage movements of the bridge, resulting in large moments in the piers and high forces being transferred to the pier foundations. To accommodate the creep and shrinkage movements a pier depth of 2.4m was required to provide the required flexibility and minimise the forces transferred to the foundations.

The compromise solution was a 2.8m deep pier section. To limit the tensile stresses in the piers during cantilever construction 52 tonnes of kentledge was used to counter balance the leading cantilever. To limit the moments in the piers and the forces transferred to the foundations, the tips of the cantilevers in span 3 (mid bridge final closure) were jacked apart 60mm to compensate for some of the predicted creep and shrinkage movements. The details of the midspan jacking are provided in section 2.4.

2.3 Superstructure

The superstructure consists of a variable depth box girders illustrated in figure 3. The box girder was designed using the provisions of the Austroads Bridge Design Code for moment design and AASHTO for shear and torsion design.

The web thickness varied from 400 near the pier to 300 for the typical internal segments. Careful consideration was given to the detailing of the reinforcement to accommodate construction tolerances and placement requirements. Variable lap hooked bars were used in the webs to accommodate the constantly changing depth of the box. A shear key joint was used for the construction joint between the segments.

![Figure 3 – Cross Section of Balanced Cantilever Bridge at Midspan](image-url)
2.4 Closure Pours

The five closures pours were constructed using different methods depending on design, construction and program requirements. The closure pours were constructed in the following order, span 2,1,4,5 & 3.

The span 2 closure pour was not on the critical path, however releasing the travellers for piers A & B cantilever construction was critical. Therefore it was not viable to use the travellers for the first closure and a custom suspended form was developed. The system consisted of a pair of strong back beams to align and clamp the cantilevers and support the exterior formwork shell.

The traditional method to construct the backspan (span 1) closure is to support the abutment segments on falsework and stitch the cantilever to these segments. Due to the unstable cliff face it was not practical to erect falsework for the construction of the abutment segments. Therefore a system to construct the segments supported off the cantilever was developed. The system utilised the strong back beams and exterior formwork shell developed for the span 2 closure to create a mini half segment traveller.

![Figure 4 – Southern Abutment Closure Falsework](image)

The span 4 closure was constructed using the southern pier A traveller with the traveller rail beams being used for aligning and clamping of the cantilevers. Due to the limited capacity of the traveller beams, kentledge and control of loads and construction activities in the opposing travellers was critical and required careful management.
The span 5 closure used a combination of the strong back beams, suspended forms and the pier A traveller. The construction sequence for the pier 5 closure was as follows

a) The diaphragm segment was constructed over the permanent bearings (50mm below final design position ref section 2.5) using the suspended formwork system with the strong back beams fixed to the end span of the incrementally launched bridge.

b) The diaphragm segment was temporarily stressed to the end of the incrementally launched bridge using temporary stress bars creating a moment connection off the end of the incrementally launched bridge.

c) The strong back beams were repositioned to support the suspended forms for segment A13 and the segment was cast

d) Segment A11 was cast using the traveller off the cantilever after the span 4 closure was completed

e) The traveller was then launched into position for the A12 closure segment and kentledge was added to the cantilever

f) The traveller rail beams were launched across the A12 closure segment and stressed down just prior to commencing the concrete pour to minimise the forces in the traveller rail beams due to thermal variations. The segment was poured late in the evening when the temperatures had stabilised. Due to the limited strength and stiffness of the traveller beams it was necessary to progressively remove the kentledge on the cantilever as the closure segment was cast to avoid overstressing the traveller rail beams

g) The following morning the continuity prestress was stressed and the connection to the incrementally launched bridge was released. The remaining continuity tendons were stressed as the closure segment concrete gained strength. Finally the diaphragm segment was jacked into its final position and the bearings grouted.

Figure 5 – Details of Falsework for Segment A13
The final span 3 closure involved the jacking apart of the two halves of the bridge prior to casting the final closure segment. A number of options were investigated for the midspan jacking. The initial option was to jack in the four corners of the box section and bury the struts into the section, similar to the approach used on Mooney Mooney Bridge. However the webs were not thick enough to accommodate the required strut dimensions, reinforcement and cover. Other options considered using corbels or brackets to support the struts external to the box section.

The final solution involved jacking off the top flange in the middle of the box. Rather than trying to cast the struts into the thin top flange, a blockout was left around the struts. Once the closure segment was cast, the struts were removed, the continuity tendons were stressed and the blockout completed.

2.5 Vertical Jacking

The construction method for casting the backspan closure segments resulted in, the majority of the self weight of the closures segments being taken by the cantilever. Therefore, insufficient permanent load was on the bearings to prevent uplift under certain load combinations. To overcome the problem, the ends of the cantilevers were built 50mm low and after completion of the backspan, the diaphragm segment was jacked vertically into its final position and the bearings grouted. Relaxation of the induced bearing reaction due to the effects of creep was considered in determining the amount of jacking required. Other options such as filling the box with mass concrete were considered but found to be uneconomical.

2.6 Provision for Travellers

The 75 tonne ‘NRC’ travellers were purchased second hand from overseas. The concept development of the box cross section was based on the geometry of the travellers, to minimise the amount of modifications required. The Contractors engineers modified the travellers to simplify the hold down arrangement and minimise the number of deck penetrations. The travellers were also upgraded to meet Australian safety requirements. The top and bottom tendons layouts were arranged to avoid the traveller penetration requirements.

The pier heads were sized to suit the minimum back to back traveller configuration with the rear frames overlapping and braced together. After the completion of the first segments the cross bracing between the travellers was released, the travellers launched and the rear frames installed for independent operation. The typical segments were 4.95m long with weight varying from 80 to 150 tonnes.
3. INCREMENTALLY LAUNCHED BRIDGE

3.1 General

The incrementally launched bridge consists of a 7 span, 203m long double tee post tensioned cross section. The 13.8m wide deck is constructed on a tight 150m radius. The 6m x 1.7m piers range in height from 8.1m to 20.4m. The shorter piers (piers 1 & 2) are solid and the taller piers are hollow with a constant 0.5m wall thickness. The piers are supported on bored piles or spread footings. The 1050 diameter piles ranged in length from 9 to 17m and are socketed into class 3 sandstone. Due to topography and access constraints, asymmetric pile arrangements were required to support the piers.

![Figure 6 – Elevation of Incrementally Launched Bridge](image)

![Figure 7 – Cross Section of Incrementally Launched Bridge](image)

3.2 Design Criteria

The design criteria for the incrementally launched bridge was based on the provisions of Austroads Bridge Design Code with T44 only loading. The design assumptions for launching included:

- 10% friction in the casting yard with the segments cast on slip foil
- 2-4% friction for launch bearings (lubricated PTFE over stainless steel)
- 50-70% friction between the gripper plates and the concrete

The range of friction values required calculations of upper and lower bound solutions to ensure adequate jacking and braking capacity was provided for launching.
3.3 Geometry Considerations

The geometry of the incrementally launched bridge was determined by road design criteria, construction requirements, topography, rockfall and aesthetic considerations.

The 2.5m depth of the section was chosen to match the depth of the balanced cantilever at the common pier. The outer faces of the beams were spaced to match the width of the balanced cantilever bridge. These two considerations provided a smooth transition between the two bridges.

The depth of the section and the available area for the casting yard determined the maximum spacing of the piers. The topography and rockfall clearances provided the positions for the piers and the incremental construction method required a constant radius and constant longitudinal gradient. Sight widening requirements for the 150m design radius determined the width of the bridge. A constant 3% superelevation was provided with the superelevation rollover provided in span 5 of the balanced cantilever bridge.

3.4 Launching

Two Eberspacher jacks were used to launch the bridge. The jacks consist of a vertical ram to lift and engage the structure mass and a thrust ram to push the structure. The typical cycle for the launching was

- lift the structure 10mm off the braking saddle with the vertical ram
- push the structure forward 300mm with the thrust ram
- lower the structure down on to the braking saddle
- return the jack and repeat the cycle

The launching process relies on the friction generated between the soffit of the girders and the gripper plate of the vertical jack. The friction force in turn is a function of the dead load reaction of the structure over the launch jack. Generally sufficient friction can be generated to push the structure, with exceptions being the initial and final launch stages where the dead load reaction over the launch jacks from the structure is minimal. For these situations the structure has to be pulled using stressbars attached to the structure and the launch jack.
For the final launch stage the reaction on the braking saddle also needs to be monitored to ensure sufficient friction is generated to prevent the structure from sliding back down the gradient due to the combined effects of gravity and the stored energy in the deflected piers and elastomeric bearings. During the final launch, when the pulling rods were being used, the bridge deck was back launched 50mm after the forward stroke to release the stored energy in the piers and elastomeric bearings, to ensure the friction generated by the braking saddle was adequate to hold the structure.

The measured launch bearing friction co-efficient was in the range of 2.4 to 3.0%.

3.5 Prestress

The prestress arrangement for the double tee cross section consisted of straight top and bottom tendons. The prestress was installed and stressed in the casting yard with no additional continuity prestress required. To minimise congestion and the number of couplers required, 50% of the tendons were anchored at each segment construction joint. The typical prestress for the internal span consisted of two 19 strand tendons in the bottom and four 19 strand tendons in the top of each tee section.

4. CONCLUSION

The design of the twin bridges of the Lawrence Hargrave Drive Project utilised the skills and experience of all members of the Alliance Team. The final solution, two curved back to back bridges provided a solution to the century old rockfall problem that has plagued this section of Lawrence Hargrave Drive between Clifton and Coalcliff.

This paper has highlighted some of the design features and the rationale behind the solutions adopted for the key elements of the curved balanced cantilever and incrementally launched bridges of the project.

Figure 10 – Lawrence Hargrave Drive Looking North