The Design and Construction of Alfords Point Bridge

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Synopsis

The bridge over the Georges River at Alfords Point has been duplicated with a new incrementally launched bridge.

The existing bridge was opened to traffic in 1973. Provision was made for its future duplication with the construction of the piles, pile caps and Abutments of the planned duplicate bridge when the original bridge was constructed.

This paper describes the investigations, concept development and detail design of the new bridge.

Existing Bridge

The existing bridge is a twin concrete box girder, built by erecting precast concrete segments on false work trusses and casting 100 mm wide wet joints between segments and a longitudinal closure strip in the top flange between the box girders. The box girders are 2286 mm deep.

The bridge has an 11.580 m wide traffic carriageway and a 1.2 m wide footway. Over recent years tidal traffic flow was implemented with 2 lanes in the peak flow direction and one lane in the counter flow direction.

The box girders are prestressed longitudinally and the top flange is stressed transversely. Also near the Piers the webs are prestressed vertically.

The substructure consisted of hollow Y shaped columns supported on pile caps in the river and spread footings at Piers 1 & 10. The pile caps were supported on four 1220 mm diameter permanently cased bored piles socketted into sandstone. The piles are raked 1 in 5.

The Abutments consisted of large box chambers, partially filled with soil.
The bridge was designed for HS20-44 truck and lane loading in accordance with NAASRA Highway Bridge Design Specification 1970. It was designed by the consultants Guyon, Schere, Harris & Sutherland in London and constructed by John Hollands Pty Ltd.

As part of the investigations for the new bridge a condition assessment and structural assessment of the exiting bridge and piles and pile caps for the new bridge was carried out.

The condition assessment indicated that the bridge is in very good condition with only minor damage and reinforcement corrosion in some precast skirting units of the pile caps. However, there have been a couple of maintenance issues with the bridge.

The superstructure of the existing bridge was fixed at the northern Abutment with guided and free sliding pot bearings provided on the Piers and southern Abutment. The English designers had underestimated the long term creep and shrinkage of the box girder, so the fingers of the finger plate joints no longer overlapped, even in summer. The rideability of the joint was not a problem, but during the course of the construction of the new bridge one of the deck joint combs broke loose requiring one lane of the bridge to be closed while repairs were carried out.

Also, the PTFE of the sliding contact surfaces of the original pot bearings had not been recessed. An earlier inspection had revealed that the PTFE was extruding, so the bearings had been rectified.

The analytical structural assessment indicated that the superstructure of the existing bridge was not significantly over loaded for the SM 1600 traffic loading of AS 5100. At ultimate, it was slightly under capacity for positive bending moment near mid span. At serviceability, there were tension stresses of up to 4 MPa.

Also, during construction of the new bridge it became apparent that the hollow low strength concrete stubs constructed to protect the column starter bars on the pile caps had not provided adequate corrosion protection to the reinforcement. There was some significant loss of cross sectional area of the reinforcement, especially on the northern wall of the stub due to the prevailing NE winds.
A large number of new starter bars had to be epoxied into holes cored into the pile caps.

Figure 2  Existing bridge deck joints  
Figure 3  Column starter bars

**Concept Design**

Alternative superstructure types and construction methods were evaluated for the new bridge, including steel trough girders, match cast segmental span by span construction, cast-insitu box girder and incrementally launched box girder.

To conform with current road design standards and the requirement for shared path the overall width of the new bridge is 15.36 m compared to 14.465 m for the existing bridge. A traffic carriageway 10.75 m wide and a 3.365 m wide shared path is provided on the new bridge.

The span lengths of the bridge are 33.93 m, 9 @ 41.91 and 34.02m.

From a structural engineering perspective, a single cell box girder would have been the optimal solution. This would have also allowed the adoption of a simpler pier column shape.

However, the urban designers wanted the new bridge to replicate the appearance of the existing, with twin box girders and the Y shaped Pier columns. After much debate an incrementally launched twin concrete box girder, with a depth on 2.6 m was adopted as the preferred solution.

While the cost of this option was estimated to be higher than a single cell box, a significant design issue became apparent during the course of the detail design which validated this decision.
On recent design and construct contracts for incrementally launched box girders in NSW, some contractors had opted to eliminate the deflected web prestressing by increasing the amount of concentric launch prestressing to satisfy final in-service conditions.

However, consistent with current German practice, it is RTA preferred practice to provide deflected web tendons in addition to the concentric prestress. The advantages are seen as follows:

- Reduced quantity of prestressing steel
- Due to the load balancing effect of the deflected tendons, the shear force to be carried by the concrete webs of the box girder is reduced
- As the eccentricity of the launching bearing reactions from the centreline of the webs must be minimised, reasonably thick webs needs to be provided. These webs can accommodate the deflected tendon prestressing ducts.
- Installing and stressing the web tendons can often take place while the diaphragms are being cast and the permanent bearings are being installed or adjusted, so there is no significant disadvantage in terms of construction time.
• The hog due the deflected tendons at least partially compensates for the self weight deflection of the box girder.

**Method of Launching**

The bridge is straight and is located on sag curve between grades of -4.17% and 6.7%. The longitudinal grade at the launching Abutment B is 6.5% and at Abutment A -2.3%.

The superstructure was launched from the southern (Menai) Abutment B end due to larger available working area. Also it allowed construction traffic to merge back into the traffic on Alfords Point Road at the top of the hill.

![Girder Launching](image1)

**Figure 5  Girder Launching**

Similar to the Woronora River Bridge, the launching system consisted of two pairs of heavy lift jacks travelling along two fixed tendons between the Abutment and the casting yard.

The heavy lift jacks, arranged nose to nose to bear against a “braking” pin inserted through block outs in the top and bottom flanges of the box girders, travel along the fixed tendons. An outline of the system is shown in Figure 7.

![Launching jacks](image2)

**Figure 6  Launching jacks**
Excluding temperature effects the total maximum braking force of required during the launch is 4800 kN and the maximum jacking force is 2000 kN. Launching downhill appears to be a good option for the construction of incrementally launched bridges as the slope component of the bearing reaction and the friction force act in opposite directions and reduce the braking force and the longitudinal forces acting on the Piers.

However, it is imperative that during the casting and launching cycles the end of the completed superstructure is held stationary near the end of the casting yard to prevent cracking at the new segment interface. Thermal contraction can result in a superstructure being pulled uphill. An ultimate reduction in superstructure temperature of 12.5°C during any launching cycle was adopted for design. This increases the braking force by 2300 kN.

Two “braking” saddles were provided at the launching Abutment to assist in holding the end of the superstructure stationary between launches and while the heavy lift jacks and braking pins were relocated for the subsequent launch. The braking saddles were operated by jacking a contact plate onto the soffit of the bridge superstructure to develop sufficient friction to hold the superstructure.

A pair of launching noses was assumed for design. Design parameters for the launching noses were provided. A steel end frame was detailed to allow launching of the last segment of the superstructure.
Casting Bed

As the casting bed was to be located in a cutting on rock, the conforming design assumed continuous concrete spline beams under each web, similar to that used on the Woronora River Bridge and the twin T superstructure of the Seacliff Bridge.

However, the Contractor opted for the more conventional discrete support casting bed with supports located under each web at the front face and middle of the bed, and at midway between the launching Abutment and casting yard to facilitate the launching of the first and last segments. The formwork was supported on longitudinal steel beams that spanned between the discrete supports.

The superstructure was generally constructed with 2 segments per span. Each segment was poured in two or three stages: the bottom flanges and webs of each box girder and then the top flange.

Detail Design Parameters

The bridge was designed to AS 5100 Bridge Design for SM 1600 traffic loading.

More conservative provisions than those required in AS 5100 were adopted for crack control during launching. No de-compression of the box girder was allowed due to self weight effects only during launching and a maximum tensile stress of $0.5\sqrt{f'_c}$ when considering additional effects of differential temperature and bearing level tolerances.

Shear and torsion design for the box girder was carried out in accordance with the AASHTO LRFD Bridge Design Specification but with AS 5100 capacity reduction factors. The combination of transverse bending of the box girder cross section in combination with shear was considered when calculating the stirrup reinforcement requirements.

As previously mentioned, a 12.5°C reduction in temperature of the box girder was assumed during any one launching cycle for calculating the longitudinal forces on the Piers and the maximum braking force.

Ultimate coefficients of friction of 0% and 4% were used to determine maximum launching and braking force and the longitudinal forces on the Pier during launching.

Foundation Evaluation

Detailed evaluation of the existing bridge foundations to support the heavier superstructure and higher traffic loadings was undertaken. New bore holes were drilled and the cores correlated to the original foundation logs to allow a more accurate assessment of the load carrying capacity of the existing piles.

The new columns were made hollow to match the existing and minimize their weight. The walls of the hollow column sit directly over the piles, so no strengthening of the pile caps was required.
Bearing Design and Contractor Modifications

For incrementally launched bridges, it is conventional practice to launch over temporary bearings and replace them with pot bearings on completion of launching.

Following the success of launching over laminated elastomeric bearings on the Woronora River Bridge, elastomeric bearings were adopted for this bridge design. Launching over elastomeric bearings has the advantage that the longitudinal force acting on the Piers due to thermal contraction is reduced due to the low shear stiffness of the bearings. Also, in their final configuration it allowed the superstructure to be elastically fixed at Piers 4, 5, 6 & 7 with smaller deck joints required at each Abutment.

At the Abutments and Piers, where the shear deflection capacity of the elastomeric bearings was not sufficient to accommodate the long term creep and shrinkage deflections and thermal movements, a stainless steel/PTFE sliding contact surface was incorporated into the bearings.

The typical elastically restrained bearing set under each web consisted of a pair of AS 09:12:07R bearings with associated attachment and keeper plates. The advantage of the twin box girder design alluded to above, is that with a single cell box girder the heavier reactions would not have allowed the use of elastomeric bearings.

The Contractor proposed several modifications to the conforming design.

Rather than using a twin set of launching noses (4 girders), the contractor proposed to use an existing single launching nose with one steel girder attached to the inside web of each box girder. By strengthening and stiffening the end of the superstructure and modifying the attachment details, it was possible to use the existing launching nose.

The footings at Piers 1 & 10 were at a considerable depth below existing ground level. To avoid the need for a large excavation at Pier 10, the contractor opted to
redesign the Pier with piles around the outside of the existing footing, a pile cap above the existing footing and a solid column.

Also in lieu of cast-in-place concrete parapets the contractor adopted precast parapet shells filled with insitu concrete.

All these modifications were checked and accepted by the RTA.

**Design Team**

The concept and detail design for the new bridge undertaken by Bridge Engineering in the RTA. An external proof check of the design was carried out by Taylor & Herbert Consultants Pty Ltd.

The Contractors design modifications undertaken by Maunsells Pty Ltd.

The contractor, Abigroup Pty Ltd, regularly achieved a weekly cycle for the construction one superstructure segment, so that from the commencement of launching of the girder to completion of the bridge taking took less than 12 months.

![Completed bridge](image)

**Figure 9 Completed bridge**

**References**