SYNOPSIS
The Coopernook to Herons Creek Alliance was formed with the New South Wales Roads and Traffic Authority (RTA), Parsons Brinckerhoff and Thiess Contractors to design and construct the upgrade of 32.7 km of the Pacific Highway to dual carriageway between Coopernook and Herons Creek on the NSW Mid-North Coast. The upgrade works involved the construction of fifteen new bridges, including two major river crossings over the Stewarts River and the Camden Haven River, which will duplicate the existing bridges. The bridge over the Stewarts River is 274 m long and comprises eight spans of 30.5 m, 6 x 38.3 m and 30.8 m, while the bridge over the Camden Haven River is 174 m long and comprises six spans of 29.0 m. Both bridges are post-tensioned, single cell, trapezoidal box concrete girders, which are designed to be constructed using the incrementally launched method. This paper discusses the site and environmental constraints, the options considered, the challenges encountered and describes the solution adopted for these bridges.

1. INTRODUCTION
The new northbound bridges over the Stewarts River and the Camden Haven River are recent examples of the incrementally launched method of construction. This paper discusses the reasons this method of construction was chosen by the Coopernook to Herons Creek Alliance by investigating the site, and the environmental and other constraints imposed upon the construction. It also looks at the options considered and the challenges encountered during design, and outlines the solution adopted for these bridges. Both bridges are currently under construction and will be open to traffic when the Coopernook to Herons Creek section of the Pacific Highway is completed at the end of 2009.

2. BACKGROUND
Since 1995, the Roads and Traffic Authority of New South Wales (RTA) has spent over $3.9 billion to upgrade the Pacific Highway between Sydney and Brisbane to dual carriageway as part of the National Land Transport Network. The section of road between Coopernook and Herons Creek on the New South Wales Mid-North Coast is the longest section of road upgrade to be carried out by the RTA on the Pacific Highway. When this section of road is built, 52% of the Pacific Highway upgrade program will be complete, leaving a further 324 km of highway yet to be upgraded (source: RTA Website, February 2009).

In response to the RTA’s request for proposal, and after a series of interviews and workshops, the RTA formed an alliance project team with Parsons Brinckerhoff Australia Pty Ltd (PB) and Thiess Pty Ltd (Thiess) in November 2007. The Alliance
was given the task of designing and constructing the 32.7 km dual carriageway upgrade of the Pacific Highway by December 2009. The upgrade work, which provides dual carriageway, 110 kph design speed highway, comprises extensive earthworks, ground improvement works, drainage, service relocation, property adjustments, concrete pavements, retaining walls, culverts, and fifteen separate bridge structures. The bridge structures included two rail crossings, three grade-separated interchanges, a number of creek and property access crossings and two major river crossings. This section of Pacific Highway is jointly funded by the NSW State and Federal Governments at a total cost of approximately $500 million (AUD).

The two river crossings include duplicating the bridges over the Stewarts River and the Camden Haven River. The new bridge over the Stewarts River is 274 m long, comprising eight spans of 30.5 m, 6 x 38.3 m and 30.8 m. It has a 2.4 metre deep, post-tensioned, single cell, trapezoidal, box concrete girder with an 11.5 metre wide carriageway, and is supported on reinforced concrete piers and bored, cast-in-place, reinforced concrete piles. The new bridge over the Camden Haven River is 174 m long, comprising six spans of 29.0 m. It has a 1.6 metre deep, post-tensioned, single cell, trapezoidal box concrete girder with an 11.5 metre wide carriageway and is supported on reinforced concrete piers and precast, prestressed, composite piles. The new bridges will form the northbound carriageways while the existing bridges, which currently carry traffic in both directions, will become the southbound carriageway. Both bridges are designed to be constructed using the incrementally launched method, adopting the Eberspächer jacking system.

The design of the bridges has involved close collaboration of both the non-owner and owner participants of the Alliance. Non-alliance parties involved in the design included Conybeare Morrison - urban design, Wyche Consulting - specialist advice on incremental launch design, and Parsons Brinckerhoff (New York Office) - external proof checking. On completion, the bridges will be owned and maintained by the RTA.

3. PROJECT BRIDGE PERFORMANCE CRITERIA

The bridges are required to comply with the requirements of AS5100-2004 Bridge Design, the Alliance Scope Document (including Appendix 14 – Structural Design Criteria) and the RTA Bridge Policy Manual. The environmental impact statements for the upgrade work also stipulated specific requirements for the bridges, including pier and abutment locations, urban design requirements and fauna habitat requirements.

The Alliance Scope document required that the bridge deck was to comprise two 3.5 metre wide traffic lanes, a 2.5 metre wide outside shoulder and a 1.0 metre wide median shoulder. A further requirement of the document is that provision be made for an additional third lane in each direction to cater for future traffic volume increases. As the cost of widening bridges can be prohibitive, the Alliance resolved to provide an 11.5 metre wide carriageway, which could accommodate three lanes with 0.5 metre wide shoulders at some point in the future. The traffic loading adopted in the design of both bridges was three lanes of the SM1600 traffic loading and a single HLP400 heavy loaded platform located within 1.0 either side of the bridge centreline. Traffic barriers on the bridge were designed as medium performance level.
The bridge superstructures and substructures are based on a Bridge Earthquake Design Category of BEDC-3. This category requires that structural detailing of the reinforcement be in accordance with Section 14.7 of AS5100.2, which stipulates the minimum ductility requirements specified in AS 5100.5 are adopted. As full blade wall piers were used, to meet urban design and construction requirements, the ductility requirements of the standard would have resulted in excessive amounts of reinforcement. To minimise the need for excessive reinforcement steel, a structural response factor (Rf) of 2.0 was adopted for piers in the transverse direction, enabling significant reduction in the amount of reinforcing steel without compromising performance.

Piers were designed for strength based on the applied loads, with the estimated maximum scour in the riverbed or flood plains based on the waterway design. The design scour depth for the Camden Haven River was calculated at approximately RL−10.0, 7 m below the current riverbed and for the Stewarts River at approximately RL−1.5, 2.6 m below the flood plain. For the bridge over the Camden Haven River, which is a navigable waterway, the piers were designed to withstand impact by a 5 tonne displacement vessel travelling at 5 m per second (ULS).

The Alliance Scope requires that the bridge bearings can be replaced without closing the bridge to traffic. Initially the intention was that no reduction in the traffic loading be allowed for bearing replacement; however, the RTA bridge technical direction BTD 2007/12 issued in December 2007, allowed the shoulders to be closed to traffic, the traffic speed reduced to 40 km/hour and the exclusion of HLP vehicles.

The design requirements during the construction stage include a 0.5 kPa live load on the bridge deck and 5 kN/m on the launch nose girders. Differential settlement effects are limited to 20 mm between any two adjacent piers or abutments, and wind loads and differential temperature effects were limited to 70% of AS5100 requirements. A temporary bearing friction factor of 5% was used to calculate launching forces and design of abutments and piers.

In accordance with AS5100-2004 and the Alliance Scope, the design life for bridges was specified as 100 years.

A further requirement for the design of the bridges is that the existing bridges, specifically the abutments, are not damaged with the construction of the new bridges and approach embankments. This was of specific concern due to the soft soil conditions in the flood plains and riverbeds, and the height of the approach embankments. The Alliance designed a series of ground improvement works to limit settlement of the ground in the vicinity of the abutments and undertook extensive modelling of the soil-structure interaction between the new works and the existing structure. This aspect of the design is not discussed in depth in this paper.

4. DURABILITY

The durability of the bridges is addressed in a report that covers all aspects of the project. It identifies a number of factors that influence the durability of the Stewarts River and Camden Haven Bridges, including:
• the Exposure Classification in accordance with AS5100.5
• chloride ions in the rivers and groundwater, including exposure of the Camden Haven River Bridge piers and pilecaps to the tidal/splash zone
• potential acid-sulphide soils

**Exposure classification**

All substructure elements on the bridges were identified as B2 Exposure Classification to AS5100.5-2004 based on Note 6 to Table 4.3, which states that a structure is in a “Coastal Zone” if it is “…immediately over or adjacent to… small saltwater bays, estuaries and rivers…” While both bridge sites are located more than 1 km from Watson Taylor Lake, which is the nearest large expanse of saltwater, according to the Moorland To Herons Creek Environmental Impact Statement (ARUP, 2007) both the Stewarts River and the Camden Haven River are classified as marine waters.

The superstructure elements on the bridges, on the other hand, have been identified as B1 Exposure Classification to AS5100.5-2004. Being in the same classified environment as the substructure, and strictly in accordance with Table 4.3, the superstructure should be B2 exposure classification. However, the Alliance carried out a series of tests on the existing adjacent bridges to determine the chloride ion profile for bridge decks. The test reports by Mahaffey Associates found that after over 30 years of exposure, the amount of surface chloride in both bridges was sufficiently low to recommend a B1 Exposure Classification.

**Tidal/splash zone treatment**

The Alliance Scope document required all reinforced concrete in the tidal/splash zone to be reinforced with stainless steel, such that the “…Stainless steel reinforcement will extend above and below the tidal zone for a distance that is appropriate for the lap splicing of the stainless steel reinforcement with the carbon steel, except that, for a pile cap where the soffit of the pile cap is no higher than 0.5 m below MLWSL and where the edge distance to the pile is at least 200 mm and the pile reinforcement extends no closer than 250 mm from the top of the pile cap, carbon steel pile reinforcement may be anchored directly into the pile cap.”

To define the extent of the tidal/splash zone the Alliance relied on testing of core samples taken from the adjacent bridge to accurately assess the height of the zone. It was found that the stainless steel reinforcement could be terminated directly above the pilecap.

**Acid sulphate soil**

Extensive testing for acid sulphate soils was carried out over the entire length of the project and the results recorded in the Durability Assessment Report. Pier 7 of the Stewarts River Bridge was found to be in marginal acid sulphate soil with a measured soil pH of less than 4.5. The pilecap concrete was reclassified as Exposure Classification U and the durability reassessed by applying the recommendations of the RTA ASS guidelines (RTA, June 1997). Design information for the concrete work for this pier was prepared separately in Annexure A of the RTA ConcreteSpecification B80. Table 1 summarises the exposure classifications, concrete strengths and covers adopted on the bridges.
Table 1: Concrete exposure classifications and covers

<table>
<thead>
<tr>
<th>Element</th>
<th>Location</th>
<th>Exposure classification</th>
<th>Concrete grade</th>
<th>Nominal cover (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Superstructure</td>
<td>Internal face</td>
<td>A</td>
<td>S50</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>External face</td>
<td>B1</td>
<td>S50</td>
<td>45</td>
</tr>
<tr>
<td>Substructure</td>
<td>Abutments</td>
<td>B2</td>
<td>S40</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>Pier columns (river)</td>
<td>B2/C*</td>
<td>S50</td>
<td>70</td>
</tr>
<tr>
<td></td>
<td>Pier columns (floodplain)</td>
<td>B2</td>
<td>S40</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>Pilecaps (river)</td>
<td>C*</td>
<td>S50</td>
<td>70</td>
</tr>
<tr>
<td></td>
<td>Pilecaps (floodplain)</td>
<td>B2</td>
<td>S40</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>Pilecaps (floodplain/ASS)</td>
<td>U</td>
<td>S40</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>Precast piles</td>
<td>B2</td>
<td>S50</td>
<td>70</td>
</tr>
<tr>
<td></td>
<td>Cast-in-place piles</td>
<td>B2</td>
<td>S50</td>
<td>85</td>
</tr>
</tbody>
</table>

* Stainless steel grade 2205 reinforcement was used in river pilecaps and pier column starter bars

5. CONCEPT DEVELOPMENT

The concept development stage of the project followed the two-stage process typical of Alliance agreements. The first stage involved producing a concept design so works could be costed and an overall Total Outturn Cost (TOC) for the project produced. Only once the TOC was agreed by the RTA, could the detail design work proceed. As the design and construction period for the project was extremely short, concept development for the larger bridges commenced at a very early stage, with close collaboration between the design team, the construction team and the peer review team (RTA Bridge Engineering).

The reference design and constraints for the bridges as indicated in the environmental impact statement (EIS) included the following:

**Stewarts River Bridge**

- 8 x 38 m spans
- piers to line up with the piers of the existing bridge (it should be noted the concept design had end spans longer than those on the existing bridge to minimise the effects of embankment overburden on the existing abutments)
- superstructure did not need to match that of the existing bridge (i.e. the use of super-T girders was not excluded).

**Camden Haven River Bridge**

- 6 x 29 m spans
- piers to line up with the piers of the existing bridge (again the concept design had end spans longer than those on the existing bridge to minimise the effects of embankment overburden on the existing abutments)
• the bridge superstructure was to match that of the existing bridge (a voided slab, 1100 m deep with a curved soffit).

In addition to the constraint imposed by the EIS, there were a number of other issues influencing the choice of structure used on the bridges. These included:

• The cost of constructing a voided slab for Camden Haven River Bridge – while 30 years ago it may have been cost-effective to construct a cast-in-place concrete deck on formwork over water, this was no longer considered viable.

• If a voided slab were adopted for Camden Haven River, it would have to be deeper due to the increased loading requirements and wider to cater for the future traffic lane (the existing bridge was designed for two lanes of T44 loading). In addition, the medium performance traffic barriers would detract aesthetically from the form intended by the urban designers who imposed the constraints in the EIS.

• While Super-T girders spanning 38 m were feasible for Stewarts River Bridge, they would be heavy, and the soft soil under the flood plain would make crane access very difficult and costly.

Various options were considered for each bridge in an attempt to comply with the constraints and requirements. For Camden Haven River these included super-T girder spans, an incrementally launched voided slab deck and a segmental construction. All these options were eventually ruled out on the basis of urban design or construction cost. For Stewarts River the options considered included super-T girders (both simply supported and continuous over the piers) and precast segmental box similar to the existing bridge. As urban design was a major issue in the decision of bridge structure, a number of digital models were produced by the Alliance urban designers, Conybeare Morrison, to assist and illustrate the relative merits or otherwise of one design over another (refer to Figure 1).

After significant modelling of the urban design aspects of both bridges, it was agreed with the RTA (Pacific Highway Office) that the requirements of the EIS could be revised to accept a trapezoidal box girder at Camden Haven River Bridge provided a similar structural form was adopted at Stewarts River. The loss of the curved soffit voided slab bridge at the Camden Haven River would be balanced by using a more attractive structural form of a single cell, trapezoidal, concrete box girder rather than super-T girders at the Stewarts River.

![Figure 1: Digital model of the new Camden Haven River Bridge](image-url)
From a cost perspective, the concrete box girder over the Camden Haven River was cheaper and more efficient than the voided slab, while the concrete box girder over the Stewarts River was effectively cost neutral. It should be noted that costing against the box girder options assumed these bridges would be constructed using the incrementally launched method. A major factor in the cost-effectiveness of this form of construction was that the Alliance construction team, Thiess, had recently completed a similar incrementally launched bridge over the Karuah River, and owned much of the specialist equipment required to carry out this method construction. Without this factored into the costs, it is unlikely that bridges 274 m and 174 m in length, constructed using the incrementally launched method, could have been justified economically. The equipment available to the Alliance construction team included:

- casting bed formwork
- launch nose girders (2 sets)
- Eberspächer launch jacks (2 sets)
- temporary bearings and side guides (more than enough for the two bridges to be launched concurrently)
- pulling and locking frames
- construction expertise in this form of construction.

Once it was confirmed and accepted that both bridges would be constructed using the incrementally launched method, a number of refinements were made to simplify construction, mainly in reducing the cycle time for the segment construction. These included adopting concentric prestress, maintaining a constant thickness bottom slab, designing the overall dimensions of the box girders so that minimal changes were needed to the existing casting bed formwork, launch nose beams and other construction equipment.

The concept design of both incrementally launched bridges was based on concentric only stressing in the box girders. While this resulted in some loss of efficiency in the box section, the benefits were achieved in that there was no second stage stressing operation and no need for deviation and anchor blocks inside the box. This design philosophy did not comply with Section 6.11(c) of AS5100.2 which requires “no sag deflection occurs under permanent loads”, however, approval was obtained by the Alliance peer review team on the basis that the long term deflection under permanent loads was sufficient small and the philosophy adopted. The final design for the Camden Haven River bridge did, however, include a single draped tendon in each web to minimise permanent deflections in the end spans, mainly due to the span configuration with the length of the end spans equal to that of the internal spans. These draped tendons are installed for the full length of the bridge, once it is in its final position and stressed from both ends.

6. TEMPORARY WORKS AND CONSTRUCTION SEQUENCE

The design of the bridge was based on a construction sequence devised by the Alliance design and construction teams to suit the available construction equipment. The construction sequence for both bridges involves casting the box girder in
segments, typically half the length of the internal spans, in specially prepared casting beds located behind the launch abutments. A launch nose girder is connected to the leading segment of each bridge to minimise stresses in the box girder during launching. After each segment is cast, it is stressed to the preceding segment, and box girder and launch nose girder assembly is launched out on temporary bearings, using specialist jacks fixed to the launch abutment. This sequence continues until all segments are in place. The launch nose girders are then removed, the temporary bearings replaced with permanent bearing, and the bridgework completed.

Figure 2 illustrates the construction sequence for the Stewarts River Bridge.

Figure 2: Construction sequence for the Stewarts River Bridge

The process of casting each segment is based on a seven-day cycle. To achieve this, concreting completed in two pours. Firstly the bottom flange and webs are cast, followed by the top flange and cantilevered deck. While one segment is being poured, the reinforcement for the following segment is being fixed in a reinforcement jig directly behind the casting bed. Once concreting of a segment is complete and a minimum strength achieved, the segment is stressed to the preceding segment by stressing 50% of the total tendons at the segment joint. The remaining 50% of the tendons carry through to the next segment joint. This method of staggering the stressing is adopted to minimise the number of anchorages at each joint, resulting in thinner flange thicknesses and a quicker construction cycle.

The construction sequence is designed such that there is no need for intermediate support between the casting bed and the launch abutment. The casting bed
comprises driven, piled foundations and cast-in-place, reinforced concrete pile caps and plinths. These support a series of steel spine beams and a specially fabricated formwork system that is lowered when the box girder and launch nose girder assembly is launched. Internal formwork for the top flange is specially fabricated for each bridge and designed with folding sections to enable the formwork to be withdrawn between the internal diaphragm thickenings at pier support locations.

The launch nose girders each comprise two fabricated steel plate I-girders connected with a series of bracing frames. Both launch nose girders had been used on previous launched bridge constructions at least two or three times. Modifications to the launch nose girders are limited to minor changes to the girder end plates and fabrication of new side guide beams. In the case of the girders for the Stewarts River Bridge, the new side guide beams are fabricated to suit the helical geometry of the bridge.

The bridges are designed to be launched using the Eberspächer jacking system, which relies on the friction between the jack and the soffit of the box girder to transfer the jacking force. For the first and last launch sequences, it is therefore necessary to provide either pulling rods attached to the launch nose girders or a fabricated pulling frame connected to the rear of the last segment. The pulling frame is designed to fit both bridges and also to be used as a locking device to ensure the bridge does not move under temperature variations, once the completed girder is in place and before the permanent bearings are installed. Figure 3 illustrates the final launch construction procedure for the Stewarts River Bridge using the pulling frame.

![Diagram](image)

*Figure 3: Final launch sequence for the Stewarts River Bridge*

7. ANALYSIS METHODOLOGY

Analysis of the superstructure was carried out using TANGO bridge design software package. The software calculates stresses created by the various construction
stages, such as creep and shrinkage effect and the launching sequence. The final results were the combination of all the stresses obtained at each stage of analysis. Other design forces due to seismic loadings, wind loads, global uniform temperature and local thermal gradient effect were included in the computer frame analysis. The combined final results were used to check against the capacity of the box girder, both in serviceability and ultimate limit conditions.

A separate simplified beam model for the superstructure was also set up using MICROSTRAN software to obtain the torsional effect due to the curve of the bridge in plan and the maximum traffic load at the edge of the cantilever. The webs and diaphragms at the piers and abutments were designed based on this torsional force, in combination with the vertical shear force.

The effect of load distributions in the transverse direction of the box girder was investigated by using a 3D finite element model set up in SAM bridge design software package. The process included setting up a plate model with the appropriate structure geometry. Only two spans were modelled for this purpose and an influence surface generated for the bridge deck. The influence surface was then used to identify and compile the critical loading patterns for the structure.

Loads on the bearings and substructures were obtained by combining the results obtained from the analyses of the superstructure with various loadings, such as traffic, wind, braking and seismic effects. The longitudinal horizontal forces for the piers and Abutment B were established from the frictional resistance provided by the slide guiding bearings, which is typically taken as 5% of the vertical reaction.

The pile group analysis software PIGLET was used to analyse the piles with different load combinations and ground profiles. The results obtained from the analysis were used to determine the reinforcement for each pile.

8. BRIDGE GEOMETRY AND THREE-DIMENSIONAL MODELLING

Three-dimensional AutoCAD modelling was prepared for all bridges on the Coopernook to Herons Creek project. This was especially relevant to the incrementally launched bridges, where the set-out of the substructure is critical to ensure the girders are not over-stressed during the launch process.

The Stewarts Bridge has an alignment that rises constantly from the southern abutment to the northern abutment, and is on a horizontal plan radius. That is, the bridge centreline follows a helical path. The three-dimensional model was used to accurately set out all aspects of the bridge deck and substructures, as well as the launching sequence. The Camden Haven River Bridge, on the other hand, has a straight horizontal and vertical alignment.

9. BRIDGE OVER THE STEWARTS RIVER

The existing bridge carrying the Pacific Highway over the Stewarts River, constructed in 1981, comprises a precast segmental concrete box girder with 30.8 m end spans and six 38.0 m internal spans. The bridge is located on a low lying flood plain with only the one span over the permanent waterway of the Stewarts River.
The new northbound bridge over Stewarts River adopts a similar geometrical layout to that of the existing bridge, with two 30.8 m end spans and six 38.35 m internal spans, resulting in an overall length of deck of 294.7 m. It is located approximately 19 m, centre to centre, west of the existing bridge.

The overall width of the superstructure is 12.91 m and it accommodates an 11.5 m carriageway, made up of two 3.5 m traffic lanes, a 2.0 m median shoulder and 2.5 m outside shoulder. The carriageway width was set to allow for a future three-lane arrangement by reducing the shoulder widths. The bridge alignment consists of a 2119 m constant plan radius, a constant gradient of approximately 1.327% and a cross fall of 3%. Figure 4 shows an elevation and typical cross-sections through the new and the existing Stewarts River Bridges.

**Figure 4: Elevation and typical sections through the Stewarts River Bridge**

**Geotechnical conditions**

Based on the geotechnical investigation, the soil profile at the Stewarts River site comprises soft alluvium layers consisting of clayey silt and silty clay to varying depths of 4–5 m overlying approximately 5 m of stiff to very stiff sandy/silty clay, again overlying another 5 m of sub-rounded to sub-angular gravelly, sandy clay. Extremely weathered (extremely low strength) siltstone/sandstone bedrock is typically encountered from RL–14 to –16 metres. Below this level, the rock quality and strength gradually improves with depth.

Table 2 summarises the geotechnical conditions at the Stewarts River Bridge site.
Table 2: Geotechnical conditions at the Stewarts River Bridge site

<table>
<thead>
<tr>
<th>Bridge Element</th>
<th>Borehole</th>
<th>Top of Class R5 Rock</th>
<th>Top of Class R4 Rock</th>
<th>Top of Class R3 Rock</th>
<th>Top of Class R2 Rock</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RL (m)</td>
<td>RL (m)</td>
<td>RL (m)</td>
<td>RL (m)</td>
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<tr>
<td>Abutment A</td>
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<td>1.4</td>
<td>-13.3</td>
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</tr>
</tbody>
</table>

Bridge substructure

Due to the ground condition, cast-in-place reinforced concrete piles with temporary casing were selected for the foundations. The temporary steel casing is required to support the unstable soft soil and gravel layers in the bore hole until the concrete pile is cast. 1160 mm diameter (with 1050 mm diameter socket) cast-in-place reinforced concrete piles are detailed for the launching abutment and 1000 mm diameter cast-in-place reinforced concrete piles (with 900 mm diameter socket) for all other substructure. These piles are designed to socket into R3/R2 rock to achieve the required geotechnical capacity.

The abutments comprise reinforced concrete sill beams supported on reinforced concrete cast-in-place piles designed to withstand the vertical and horizontal loads acting longitudinally and transversely to the bridge centre line. They also include a reinforced concrete back wall, which to protect the bearings against ingress of water or soil from behind the abutment, and reinforced concrete wing walls to enclose the abutment and contain the road embankment fill.

The typical pier is a single rectangular reinforced concrete section, tapered in the transverse direction. Each pier is supported by cast-in-place reinforced concrete piles and a pile cap.

Bridge superstructure

The bridge deck comprises a single cell, 2.4 metre deep, box girder with constant depth. It is designed as a concentrically prestressed section with all tendons located in either the top or bottom slabs. Steel reinforcement is added longitudinally in the tension face of the box structure to control the tensile stress increment in the extreme concrete fibres. The amount of prestressing force required was calculated to satisfy crack control under the AS5100.5 Bridge design, clause 8.6.2 (a) and (c), but with the increment in steel stress near the tension face limited to 160 MPa, as required by the Alliance Scope. The box girder is designed to be cast, prestressed and incrementally launched from the casting yard located at the southern abutment using a 27.8 metre long launching nose.
The continuous bridge structure is supported at the launch abutment by a fixed pot bearing and a sliding-guided pot bearing restrained in the longitudinal direction. Longitudinal loads, such as braking forces and seismic forces, are transferred from the superstructure to the fixed abutment through these bearings. The remaining supports consist of free-float or sliding-guided pot bearings allowing movements in the longitudinal direction due to creep, shrinkage, and effects of temperature variation.

Transverse loads, such as earthquake and wind loads, are resisted at all supports by the fixed or the sliding-guided bearings. Locations of temporary jacks for future bearing replacement and the required jacking loads are indicated on the drawings.

A fingerplate joint is detailed at the northern abutment to accommodate movements due to shrinkage, creep, thermal expansions and contractions. A 750 mm access is provided between the back of the box girder and the curtain wall at the northern abutment for maintenance purposes.

Diaphragms are provided at supports to transfer forces from the superstructure onto bearings. The end diaphragm at the first launched section is solid and three metres thick to allow for a rigid connection between the launching nose and the bridge deck. All other diaphragms are typically two metres thick and allow internal access. All diaphragms were designed and analysed with finite element analysis software package STRAND7, using solid brick elements to model the behaviour of the diaphragms.

Traffic barriers on the bridge consist of a 650 mm high concrete barrier with two steel rails above, providing an overall height of 1300 mm. The barriers have been designed as medium performance in accordance with AS5100.2. The concrete barrier and fascia were initially designed as precast concrete elements attached to the deck slab with a cast-in-place “stitch” pour; however, this was later changed to a cast-in-place section.
10. BRIDGE OVER CAMDEN HAVEN RIVER

The existing bridge carrying the Pacific Highway over the Camden Haven River, constructed in 1986, comprises a 1.1 metre deep, multicellular prestressed concrete deck with 23.5 m end spans and four 29.0 m internal spans.

The new 1.6m deep, single cell, prestressed concrete box girder for the northbound bridge over Camden River adopts a similar geometrical layout to that of the existing bridge, with two 28 m end spans and four 29 m internal spans, resulting in an overall length of deck of 197.97 m. It is located approximately 19 m, centre to centre, west of the existing bridge. The longer end spans mean the new bridge is slightly longer, with the southern abutments are approximately aligned and the northern abutments offset to allow Sunnyvale Road to pass in front of the abutment. The piers in the river are offset by approximately six metres to allow for the skew of the river relative to the bridge alignment.

Similar to the bridge over Stewarts River, the overall width of the superstructure is 12.91 m which will accommodate an 11.5 m carriageway, made up of two 3.5 m traffic lanes, a 2.0 m median shoulder and 2.5 m outside shoulder. The carriageway width was set to allow for future three-lane arrangements by reducing both shoulder widths to 0.5 m. Figure 5 shows an elevation and typical cross-sections through the new and the existing Stewarts River Bridges.

Figure 5: Elevation and typical Sections through the Camden Haven River Bridge
Geotechnical conditions

Based on the geotechnical investigation, the soil profile at the Camden Haven River site comprises 7–10 m of clayey soils, mostly silty to sandy and gravely clay, varying from soft to very stiff consistency. This overlies 10–14 m of loose to very dense, clayey, gravelly sand and sandy gravel, followed by high strength rock of varying types, including sandstone, claystone and siltstone. A lens of tuff/tuffaceous sandstone was also encountered in a number of boreholes.

Table 3 summarises the geotechnical conditions at the Camden Haven River Bridge site.

<table>
<thead>
<tr>
<th>Bridge Element</th>
<th>Borehole</th>
<th>Top of Class R5 (RL in m)</th>
<th>Top of Class R4 (RL in m)</th>
<th>Top of Class R3 (RL in m)</th>
<th>Top of Class R2 (RL in m)</th>
<th>Top of Class R1 (RL in m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abutment A</td>
<td>R_BH418 M_BH1 A_BH21</td>
<td>-23.3</td>
<td>-24.2</td>
<td>-24.5</td>
<td>-</td>
<td>-27.2</td>
</tr>
<tr>
<td>Pier 1</td>
<td>A_BH21</td>
<td>-23.3</td>
<td>-26.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Pier 2</td>
<td>R_BH419</td>
<td>-27.5</td>
<td>-28.9</td>
<td>-32.3</td>
<td>-23.4</td>
<td>-25.8</td>
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<tr>
<td>Pier 3</td>
<td>R_BH420</td>
<td>-21.5</td>
<td>-21.8</td>
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<td>-</td>
<td>-</td>
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<tr>
<td>Pier 4</td>
<td>R_BH421</td>
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<td>-24.9</td>
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<td>Abutment B</td>
<td>R_BH422 R_BH423</td>
<td>-24.1</td>
<td>-22.4</td>
<td>-29.5</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Bridge substructure

The spill-through abutments comprise of 2 m thick pilecap supported on precast, composite driven piles, designed to take vertical and horizontal loads. The launch abutment is located at the northern end of the bridge and founded on fourteen 600 mm, square precast, composite driven piles, founding on the rock layer. Longitudinal loads, such as braking loads, seismic and launching forces, are resisted by the launching abutment through the permanent bearings, or during the launch process, by the braking saddle fixed to the top of the pile cap. The non-launching southern abutment has two rows of four 600 mm, square precast, composite driven piles founding on the rock layer.

The abutments also include a curtain wall and wing walls, which not only support the embankment fill and approach slab, but also protect the bearings against soil and water ingress from behind the abutment. A precast concrete curtain wall has been detailed for the northern abutment, as there is insufficient space to provide formwork after the bridge girder has been launched into position and the launch nose girders removed. The spill-through embankment in front of the abutment is protected from erosion and scour by the rock armour on a 1:1.5 batter.

The piers comprise single columns of rectangular cross-section, with all four faces tapered. They are supported by a pile cap and a pile group consisting of ten 600 mm square, precast, composite, driven piles. In addition to the vertical loads from the
superstructure, the piers are designed for horizontal forces resulting from friction through permanent or temporary bearings, traffic loads, wind, seismic force, loads due to ship impact and forces resulting from water flow. The pile caps have been designed and detailed with stainless steel reinforcement.

Precast composite driven piles (600mm square with 310UC cast in steel section) have been adopted for the bridge substructure due to their ease of installation over water. The length of the piles means that splicing is necessary and the composite piles can easily be spliced by site welding. The design requires the piles to be socketed into R4 or harder rock to achieve the required geotechnical capacity, and also to limit the amount of total and differential long-term settlements for the bridge.

The load capacity of the driven piles was assessed using GRLWEAP software, with input of the soil data and hammer details as provided by the piling contractor. The pile group analysis software PIGLET was used to analyse the piles with different load combinations (service and ultimate) and ground profiles taking into account the effect of 1:2000 year scour. Additional analyses to investigate the ground settlement at the embankment to account for the full potential negative skin friction and down-drag loads were also included for the design of the abutment piles.

**Bridge superstructure**

The bridge deck comprises a single cell 1.6 m deep, box girder with constant depth. Similar to the Stewarts River Bridge, it is designed as a concentrically prestressed section with all tendons located in either the top or bottom slabs, except that once the launch is complete a secondary draped tendon is added to each web to minimise deflection on the end spans. The same design principle and construction methodology as the bridge over Stewarts River is applied to this bridge. The box girder is designed to be cast, pre-stressed and incrementally launched from the casting yard located at the northern abutment using a 18.0 m long launching nose.

![Figure 7: Camden Haven River Bridge – substructure construction](image-url)
The continuous bridge structure is supported at the launch abutment by a fixed pot bearing and a sliding-guided pot bearing restrained in the longitudinal direction. The remaining supports consist of free-float or sliding-guided pot bearings to allow movements due to creep, shrinkage, and effects of temperature variation in the longitudinal direction.

Similar to the Stewarts River Bridge, a fingerplate joint is detailed at the expansion abutment to accommodate movements due to shrinkage, creep, thermal expansions and contractions, with a 750 mm wide access for maintenance purposes.

A two metre thick diaphragm with an access opening has been provided at each pier locations and at the northern abutment. A three metre thick diaphragm is provided at the southern abutment to allow for a rigid connection between the launching nose and the bridge deck. All diaphragms were designed and analysed using finite element analysis software package STRAND7, using solid brick elements to model the behaviour of the diaphragms.

The medium performance traffic barriers for the Camden Haven Bridge are similar to those for the Stewarts River Bridge, comprising a 650mm high concrete barrier with two steel rails above, providing an overall height of 1300 mm. Urban design requirements resulted in a modified outside shape of fascia to be more sympathetic to the shape of the fascia on the existing bridge.

11. CONCLUSION

The 32.7 km dual carriageway upgrade of the Pacific Highway between Coopernook and Herons Creek will improve the quality of driving and reduce travel time. But most importantly, it will improve the safety on this section of road.

The new bridges over the Stewarts River and the Camden Haven River are a result of a close and successful collaboration between the Alliance participants in adopting an appropriate, innovative and cost-effective solution.

12. ACKNOWLEDGEMENTS

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