THREE RAIL BRIDGES, CONSTRUCTED IN 3 DAYS
RANGIRIRI, WAIKATO

JOHN MCNEIL¹, BEN RYDER¹, RUDOLF JOUBERT², ROBIN SCOTT³

¹ Aurecon NZ Ltd
² HEB Construction
³ KiwiRail

SUMMARY

As part of the ongoing upgrade of old timber piled bridges on the North Island Main Trunk (NIMT) KiwiRail identified three bridges (BR 279/280, 281, 282 NIMTL) to be replaced in 2012. These bridges all cross local streams of width from 16m to 42m and provide a ballast deck for twin rail lines. A collaborative approach was undertaken with a focused team bringing the design consultant, the constructor and KiwiRail together to develop a standard design for all bridges using precast prestressed concrete girders in a unique application for rail works. To allow the bridges to be completed within a 3 day rail closure an innovative construction sequence (to slide the decks into place) was successfully developed. This allowed all three bridges to be replaced at the same time within the allowed 72 hour block of line.

Figure 1. Completed Bridge 280 NIMTL
Key areas of success included.

- Developing effective access and temporary works designs.
- Fast tracking of the project delivery to meet the Christmas rail closure by using an accelerated design and consenting process.
- Development of an innovative solution using precast concrete girders.
- Design of bored concrete piles in deep soft ground conditions.
- Careful attention to safety and allowable construction clearances at the concept stage to design out the risk in working in the rail corridor.

The paper will present the background to the project, the use of in situ and precast concrete as well as the key issues associated with the design and construction of the project.

**BACKGROUND**

The original rail bridges in New Zealand were constructed at the beginning of the 1900’s with many now over 80 years old. They were typically constructed with Australian hardwood timber driven piles and timber pier caps, with tracks directly fixed to steel girders. They have performed well over many years of service, however several bridges now require increasing levels of inspection and maintenance as well as having speed restrictions imposed to limit dynamic effects on the structure. In the last 3 years KiwiRail have focused on the replacement of all timber piled bridges along the North Island Main Truck (NIMT) between Auckland and Hamilton. The replacement of the three bridges at Rangiriri leaves just one timber piled bridge on this section of track which is planned for replacement in 2014. This paper presents the solution adopted to replace Bridges 279/280, 281 & 282, which were completed as a single contract for replacement in a track closure over Christmas 2012. The requirements from KiwiRail were to;

- Replace the existing structures with robust, up to standard structures which will require minimum maintenance and inspection for a 100 year design life.
- Provide a solution which minimises the disruption to train operations.
- Complete the replacement under a single 72hr track closure in Christmas 2012
- Provide effective stakeholder consultation and develop a solution which is supported by all stakeholders and is therefore consentable.

The project was delivered as an Early Contractor Involvement (ECI) contract to allow a collaborative approach to between KiwiRail, stakeholders and the construction team.

Over the three day Block of Line (BoL) from Boxing Day 2012 all three of the existing bridges were successfully demolished and the new superstructures jacked sideways into their final position. Approach track realignments and rebuilding a level crossing on the section of track between the bridges was also undertaken in the same block of line.
LOCATION

The bridges and are located sequentially along a 1.0 km stretch of the North Island Main Trunk (NIMT) railway located approximately 1km East of Rangiriri. They cross local waterways which feed Lake Kopuera, Lake Waikare and the Waikato River. The site is near the location of the Battle of Rangiriri which took place in 1863 during the NZ land wars. It is a very sensitive site for local Iwi.

![Figure 2. Bridge locations](image)

PROCUREMENT AND CONSULTATION

For operational purposes it was decided that replacement of the bridges would need to be achieved within a limited timeframe of 14 months. This period would include planning, contract procurement, design and construction. To achieve this, KiwiRail decided to engage a contractor under an Early Contractor Involvement model. A collaborative approach between KiwiRail, and the contractor allowed for quick and effective resolution to design issues and development of robust construction methodologies.

Significant consultation was undertaken with local Iwi with a combination of informal meetings, site blessings and participation at technical workshops all contributing to developing a trusted relationship.

GEOTECHNICAL CONDITIONS

Geology

The site is blanketed by about 20m thick clayey silt and/or silty sand. The clayey silt is soft to firm with localised layers being very soft. At depths approximately 20m below the ground surface the site is underlain by dense sand and stiff to hard clay/siltstone with SPT ‘N’ more than 50.
Seismic Design Loading

The design earthquake intensity followed the requirements of NZS/AS 1170 and the New Zealand Geotechnical Society Guideline for Identification, Assessment and Mitigation of Liquefaction Hazards (July 2010). According to the seismic design earthquake provision of this code and practice standard, a generic earthquake magnitude M7.5 having a probability of exceedance less than 10% at an epicentral distance 20km away shall be assumed in liquefaction assessment.

Adopting a design life of 100 years, the seismic input parameters and the recommended design Peak Ground Acceleration PGA are summarised as follows:

Table 1. Recommended Seismic Design Input Parameters

<table>
<thead>
<tr>
<th>Level of Importance</th>
<th>Site Class</th>
<th>Spectral shape factor</th>
<th>Zone Factor</th>
<th>Return Period</th>
<th>Near Fault Factor N</th>
<th>Structural Performance Factor Sp</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>D</td>
<td>1.12</td>
<td>0.15</td>
<td>2500 years</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Table 2. Recommended Design Peak Ground Acceleration

<table>
<thead>
<tr>
<th>Type of Structure or design element</th>
<th>Return Period</th>
<th>Risk Factor R</th>
<th>PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Structures</td>
<td>2500 years</td>
<td>1.8</td>
<td>0.30g</td>
</tr>
<tr>
<td>Retaining Walls and Cut &amp; Fill slopes associated with Bridges</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

LIQUEFACTION

Soil liquefaction was identified as possible with a serious impact on both the embankment stability and the bridge structure. For practical purposes, the liquefaction evaluation procedure was separated in two categories as below:

A "sand like" liquefaction behaviour where the undrained soil response under monotonic and cyclic shearing is fundamentally applicable. In a liquefaction event, the behaviour is associated with the onset of high excess pore water pressures (usually 100%) and development of significant strains accompanied by a significant shear strength reduction. The inclusion of fines fraction and plasticity of the fines fraction can change the cyclic dynamics needed to produce a liquefied state; and

A "clay like" strain softening behaviour where the undrained response under monotonic and cyclic loading is similar to that of a clayey material. The potential for development of cyclic shear strains and strength loss are usually less than those experienced by a sandy soil, but with a strength loss highly dependent on the nature of the clayey soil (i.e. normally consolidated, lightly consolidated due to aging, thixotropy, structural bonding, etc). The low permeability of clayey soil does entail a slower dissipation of excess pore water pressures (i.e. consolidation) after the seismic event compared to the sandy materials.
The sites clay-like soils were calculated to be sensitive to liquefaction, indicative of a potential significant loss of strength to the residual undrained shear strength during a seismic event. The bridges foundations were therefore designed for the following effects:

- Post-Liquefaction Soil Reconsolidation
- Liquefaction induced pile down-drag loads or negative skin friction due to significant ground settlements.
- Pile lateral pressures due to lateral spreading

For the case of kinematic loading from lateral spreading, JRA (2002) guidelines were used which impose lateral pressures from both: (a) the liquefied layer; and (b) overlying non-liquefied soils as illustrated in Figure 3. The non-liquefied layers are considered to impose passive earth pressures, whereas the liquefied layers are considered to impose lateral pressures equal to 30% of the total overburden stress.

![Figure 3 – Idealisation of ground flow force for seismic design of bridge foundations.](image)

**HYDROLOGY AND HYDRAULIC ISSUES**

The following conditions effect the Hydrology and Hydraulic conditions at the site:

- Te Onetea Stream flows from the Waikato River into Lake Waikare and is controlled by flows through the Onetea control gate that regulates flows from the river to the lake. The Onetea control gate has a maximum flow capacity of 15m³/s for up to a 20 yr event.
- The Rangiriri Spillway near the control gate allows overflows in events larger than the Average Recurrence Interval (ARI) 20yr storm.
- Lake Waikare discharges through a control gate on the north eastern side of the lake. Lake Waikare forms part of the Waikato regions flood control measures and attenuates large return period storm flows from the Waikato river (from the rangiriri spillway) and the lakes own catchment this is achieved by increase of lake level to store this water and release of the floodwater in a controlled manner back to the Waikato river via the control gate and canal system, due to the proximity of the lake to each of the three rail bridges the lake also dictates the flood level below each of the bridges.
- The Rangiriri Stream and Kopuera Stream flows are derived from a rational method assessment, allowing for a catchment area of 3 km² and runoff coefficient varying from 0.3 (ARI 2yr event) to 0.4 (ARI 10yr event).
An allowance for the impact of climate change is included, assuming 2°C increase in average temperature over the lifetime of the bridges in accordance with Ministry for the Environment guidelines.

The 100yr flows were derived from Environment Waikato. The 50yr event flows are dominated by overflows from the Rangiriri Spillway and were derived from scaling back the 100yr spillway overflow.

The design flows and levels were calculated for 50yr and 100yr for consenting and KiwiRail approvals as well as calculating levels for 2yr and 5yr return events to provide set out levels for temporary works. Stream levels were governed by downstream lake levels and allowance for low and high existing lake levels were considered to identify worst cases.

The impact of the proposed bridges on flood levels was assessed and the flood levels compared with existing flood levels for the ARI 100yr event. The results show an increase in flood levels of no more than 35mm. Environment Waikato advised that they had no concerns with such an increase provided that there is sufficient clearance to the bridge soffit.

The bridges were set out to provide at least 1.0 m clearance to the 50yr flood and with bridges 280 and 281 providing greater than 500mm clearance to a 100 year event. Bridge 282 was identified as been inundated in a 100yr flood but with the nature of flow being back water from the Lake and a small upstream catchment with no history of debris build up at this location this was considered a preferable solution to an extensive track vertical rise.

### Table 3. Bridge flood clearances.

<table>
<thead>
<tr>
<th>Bridge</th>
<th>100yr. Flood RL.</th>
<th>Soffit RL.</th>
<th>Clearance 100yr.</th>
<th>Clearance 50yr</th>
</tr>
</thead>
<tbody>
<tr>
<td>NIMT 280</td>
<td>8.0</td>
<td>8.55</td>
<td>550mm</td>
<td>1.05 m</td>
</tr>
<tr>
<td>NIMT 281</td>
<td>7.8</td>
<td>8.55</td>
<td>750mm</td>
<td>1.25 m</td>
</tr>
<tr>
<td>NIMT 282</td>
<td>7.7</td>
<td>7.48</td>
<td>(220mm)</td>
<td>1.00 m</td>
</tr>
</tbody>
</table>

**DESIGN CRITERIA**

The bridges were designed to the requirements of the KiwiRail Network Railway Bridge Design Brief Issue 6, 2011. The primary design code referenced is the American Rail Engineering and Maintenance Association (AREMA) Manual for Railway Engineering 2009. NZ standards are also referenced in the design. The bridges were detailed for a 100 year design life.

**Loadings**

The bridges were designed for loads defined in the KiwiRail Railway Design Brief, NZTA Bridge Manual and associated New Zealand loading standards including NZS1170 Structural Design Actions.

Construction loads allowed for the placement of additional ballast and plant onto the bridge before placement in the final position.

Piers and abutments were assessed for flood water pressure to the requirements of the NZTA Bridge Manual. The bridges were designed for a 1 in 2500 ULS flood event in accordance with section 3.4.8 of the NZTA Bridge Manual. Flood loading was identified as significantly less than lateral loading from earthquake.
**Design loads ballast**

Ballast deck bridges provide the advantage of ease of maintenance allowing for both minor track realignment and also allowing track tamping and ballast clearing operations to continue along the length of the route without interruptions from bridges. They also provide a constant stiffness to the track as opposed to the original bridges which have the track directly fixed to the girders and provide a stiff transition from the approaches which are on earthworks and ballast.

For the bridges at Rangiriri the requirement for minimum ballast was reviewed as well as the vertical alignment to identify the need for future track rising. A design minimum ballast depth of 470mm was allowed for in set out of the bridge at opening and maximum ballast depth of 600mm was allowed for future loading.

**MATERIAL PARAMETERS**

<table>
<thead>
<tr>
<th>Concrete Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-stressed girders</td>
<td>50 MPa</td>
</tr>
<tr>
<td>Pre Stress - at transfer</td>
<td>35 MPa</td>
</tr>
<tr>
<td>Cast in-situ elements</td>
<td>40 MPa</td>
</tr>
</tbody>
</table>

For pre-stressed girders the following serviceably limit state (SLS) stress limits were designed for:

<table>
<thead>
<tr>
<th>Stress Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>At transfer concrete tension stress</td>
<td>to NZS3101:2006</td>
</tr>
<tr>
<td>Under full dead and live load</td>
<td>0 MPa Tension</td>
</tr>
</tbody>
</table>

**DESIGN**

**Alignment Options Considered**

The design replaced each bridge in the same location as the existing bridges and allowed all work to demolish the existing bridges and replace them to be undertaken over 72hr. A solution to build replacement bridges on a new alignment next to the existing bridges would have allowed the works to be undertaken independently of constrained track closure times. This option was not adopted for the following reasons.

- Works would be required outside the rail corridor designation and would therefore require the re-designation of a new corridor with associated uncertainty with land purchase requirements and impact on cost and program,
- There was risks of a drawn out stakeholder approval process with consideration of the alignment moving very close to an existing residential property,
- There was un-certain requirements and associated costs for significant earthworks requirements on the approaches to all bridges.
- Any pre-settlement requirements for embankments might have delayed the works.

**Track alignment**

All bridges were replaced online. The existing alignments were revised to provide improved curves and transitions and meet standard clearances between the tracks of 3.8m. This was achieved within the 72 hour block of line and was therefore limited to the extent of the existing formation allowing only 200-300mm of movement of each track.
The solution used standard precast girders and provided a wider width than the minimum required for two tracks. This allowed future horizontal realignment options of a track shift of 0.5m in either direction without any modification to the bridge. The design also considered the option to relocate girders transversely across the pier caps if a significant future realignment is required.

**Bridge form and span arrangements**

All of the bridges cross local streams with a requirement to provide cost effective solutions and minimise the impact on the environment. Te Onetea was the widest stream at 40-43 meters. Single span options were developed but were not economical. The preferred solution at Te Onetea was two 16m spans with a 2.5x2.5m culvert on the southern end to allow for continued farm access. From this the use of 16m spans for the other crossings was preferred to realise efficiencies in girder procurement.

Span length was also dictated by the existing structures. The construction method required the piers and to be constructed between the existing piers and below the existing deck prior to the demolition of the existing structure.

The deck is formed from 8 single hollow core girders which are transversely prestressed together to form 2 separate independent decks of 4 girder each.

![Figure 4. Typical cross section](image_url)

**Bored Concrete piles**

The piles were bored cast in-situ with permanent steel casings driven into hard clay.

Bored piles were adopted as they were able to provide sufficient capacity for the large vertical and lateral demands imposed on them by live and earthquake loading.

Cracked concrete section stiffness's were used for the piles with a 20% increase in calculated pile stiffness to account for the confining effect of the permanent steel jacket (Priestly et al). The casing was not included in the bending capacity of the piles.
Seismic horizontal loads

The seismic design of the bridges was based on Section 5 of the NZTA Bridge Manual. As there is a possible soil strength stiffness degradation in a large earthquake event, three probable geological profiles were modelled to allow for changing soil stiffness.

The first profile assumes the soil still has all of its original strength and stiffness and the embankments leading to the bridge provide longitudinal restraint. The other two models assume that the embankment has failed and that it no longer provides any restraint. Two soil spring stiffness’s are used in this model the original soil springs and the remoulded soil spring stiffness.

Pre stressed concrete Girders

Each deck consists of eight single hollow core girders. The girders for each half of the bridge were placed on temporary supports formed from cantilever spans of the pier headstocks. The 2 sets of four girders were transversely stressed together prior to being slid into their final position. No structural connection is provided between the two transversely stressed bridges.

Transverse stressing locations were defined to ensure there was a continuous and constant compressive stress at the shear key interface between the girders. This allowed the pre-stressed concrete to be designed with zero tension in the extreme concrete fibre both in the longitudinal and lateral direction.

The bridge superstructure is designed to be semi integral at the abutments and piers to provide a propping force to the horizontal earth pressure loads from the abutments during service through a vertical dowel at the end of each girder.

CONSTRUCTION

Works prior to the block of line

- Piles, piers and abutments were constructed outside of train clearance envelopes and under the existing bridges around operating trains.
- Precast girders were delivered to site and lifted on to the widened pier caps. Four 1 meter wider girders were placed on each side of the existing bridges and grouted and stressed together to form a four metre wide deck.
- Trails lifts and sliding the 4 sets of girders was undertaken before the block of line.

Girder placement during block of line

The key constraint for construction was to replace the bridges in the same location as the existing bridges with a 72hr planned block of line (line impassable or track closure). The block of line works included the demolition of the existing structure, the shift of the deck into its final position and undertaking of civil works for the approaches and track reinstatement.

The following construction sequence was undertaken during the track closure;

- The tracks were removed from the existing bridges
- The existing bridge decks were removed and piers cut off below deck level.
- The new girders were jacked into position on load skates
• Once in position the girders are jack off the load skates and lowered on to rubber strip bearings.
• The girders dowels were grouted into position.
• Precast abutment back walls were place and grouted in to position on the pier caps.
• The existing abutment backfill was replaced using reinforced fill.
• Ballast and tracks were placed and tamped prior to opening the line to trains.

PROJECT OUTCOMES

Proactive planning meant the design and construction team were quickly able to establish on site and begin enabling works while designs were being finalised. Once established they engaged with all stakeholders including local regulatory authorities, local landowners, Tainui and KiwiRail's regional office to ensure that they understood and managed the different groups' concerns and requirements.

Excellent planning, communication and team work meant that the block of line process was effectively managed with KiwiRail. The replacement of the bridges and associated earthworks were completed well within the 72 hour block of line period with no safety issues.

From KiwiRail's perspective, this project is regarded as an excellent success. The project achieved KiwiRail's design objectives, on time and on budget.

Figure 5. Completed bridge 280 NIMT

ACKNOWLEDGEMENTS

Client: KiwiRail
Design: Aurecon.
Construction: HEB Construction