50 YEARS of Concrete
The New Zealand Concrete Industry
CONFERENCE 2014
Wairakei Resort, Taupo 9-11 October

SOUTH RANGITIKEI RAILWAY BRIDGE - CONSTRUCTION ENGINEERING

GRAHAM FROST[1], CLIVE TILBY[2]

[1] Fletcher Construction Company
[2] Spartan Construction Group

SUMMARY

The South Rangitikei Rail Bridge is the southernmost, and longest, of three major NIMT prestressed concrete bridges that were constructed as part of the Mangaweka to Toitoi Deviation which opened in 1981.

The 315m long viaduct has six spans of up 56m and its twin leg piers that rise up to 76m above river level and are designed to “step” under large scale transverse seismic demand, making it one of the first bridge structures in New Zealand to incorporate base isolation. The falsework collapse here on 5 May 1975 and then the Karangahape Ramp A collapse one day later were the main drivers behind the MWD writing the Code of Practice for Falsework and Formwork and its Commentary.

This paper describes some of the construction challenges associated with this bridge and the engineered solutions that were developed to overcome those challenges.

BRIDGE HISTORY

The South Rangitikei Rail Bridge is the southernmost, and longest, of three viaducts that were constructed as part of the Mangaweka to Toitoi Deviation of the NIMT which started in 1973 and was opened in 1981.

The original rail line through this area was constructed as part of the NIMT in 1904. All of the route from Marton north to Taihape was built on the western of the Rangitikei River and lies predominantly on the terraces within the river valley. However, for the central section between the settlements of Mangaweka and Utiku there is almost no western river terrace. It is mostly steep hill country running into tall river cliffs. And the terrace on the eastern side of the river is deeply incised by the Kawhatau River which joins the Rangitikei in this area. So the old alignment ran along the cliff tops west of the river included a climb at each end, several tunnels and a large steel viaduct just north of Mangaweka.

By the 1960s concerns existed about the long-term stability of the land the section ran through. While the general Mangaweka area was susceptible to slips due to its steep hilly nature, rainfall and underlying geology the railway, by virtue of its cliff top nature and several tunnels, was thought to be under particular threat from erosion and collapse.

At the same time concerns were also being voiced over State Highway 1 as the steep, narrow and twisting alignment was also prone to slips. It had been built after the railway in the same area though further to the west. And naturally imposed speed restrictions were exacerbated by heavier vehicles and this created bottlenecks.

So investigations were commenced into improving both routes, including the feasibility of a joint road and rail structures on the alignment.

While SH1 ended up being reconstructed along a route that stayed on the western side of the river, the new NIMT route followed one of the joint options. It followed an almost flat grade throughout its length but required bridges for three major river crossings – all at least 70m above river level. The South Rangitikei Rail Bridge crosses the Rangitikei River about 2km north of the township of Mangaweka.

While the 1972 tender invitation called for a conventional plate girder bridge, NZ Railways encouraged alternative designs, as their preliminary studies had concluded that a prestressed concrete structure could be significantly cheaper to build. The problem was to find a viable construction procedure considering the characteristics of the site.

Codelfa Construction were awarded the contract in June 1972 on the basis of a prestressed concrete structure designed for them by Beca Carter Hollings and Ferner. See "Figure 1"

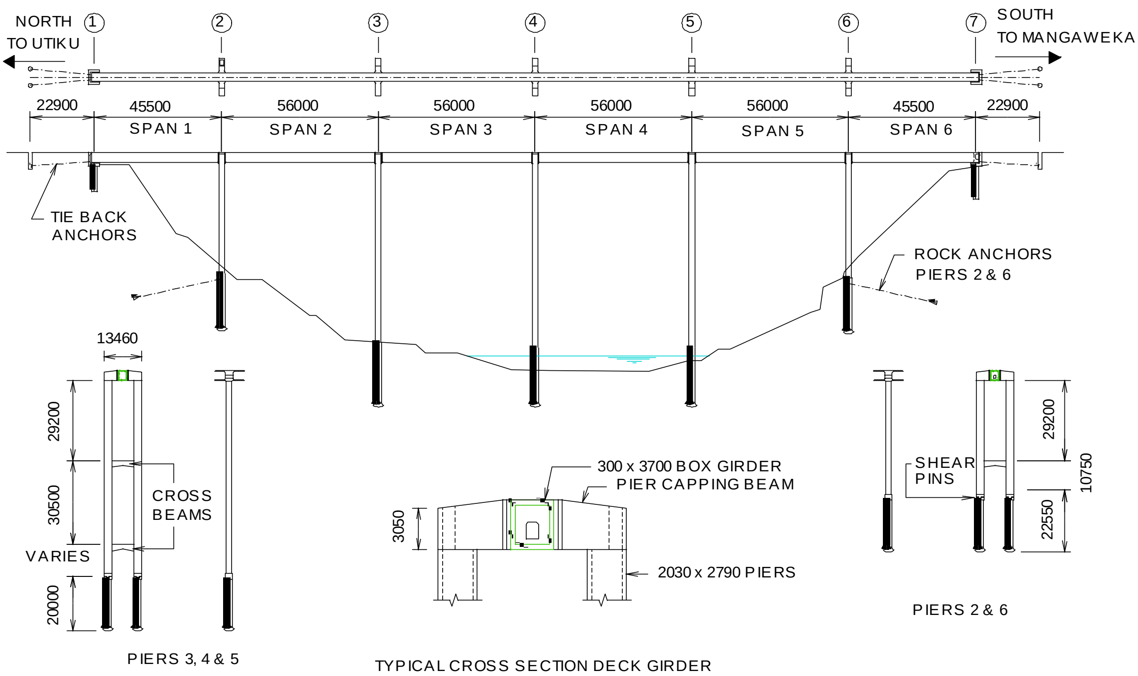

Figure 1

To overcome the problems of constructing the 46 and 56 m spans over 70 m above the river, Codelfa proposed the use of a steel truss falsework or launching girder (available from another contract) and this had a direct bearing on the chosen construction procedure.

On May 5 1975, during construction of the first bridge span, the front section of the launching girder collapsed. Refer also to a paper by Cormack (1988) on the design and construction of all three major bridges on this deviation and a previous Tilby paper (1981) with much more detail on the construction of the South Rangitikei Railway Bridge.

Work on casting of the superstructure recommenced in January 1980 using a new launching girder and with new contract conditions in respect to the design and use of the temporary works. The bridge was completed in 1981.

Figure 2
Completed bridge
looking Downstream

CONSTRUCTION ENGINEERING

The major falsework collapse at this site on May 5 1975 and then the Karangahape Ramp A collapse in Auckland one day later lead the MWD to write the Code of Practice for Falsework and Formwork and its Commentary – to help reduce the chances of such disasters in future. The extra review requirements recommended in these documents were adopted as part of the prolonged negotiations that allowed work on this project to get started after a break of almost five years.

At the time of the falsework collapse the combined pier 6 capping beam and cantilever pour, containing 170 cubic metres was nearing completion. At about 2.30 p.m., when 20 cubic metres of concrete remained to be placed, the form was felt to drop suddenly and the workmen ran immediately to the safety of the south abutment area. On inspection it was observed that two tubular web members in the girder under the cantilever section had failed by buckling and it became apparent that some load had been transferred to a new, unintended, parallel load path - through the bolted connection between the superstructure form and the piercap/crosshead form (which was independently supported on the two pier legs). A half hour of drama followed as the girder continued to settle slowly under the cantilever section and the chord members became more and more bent. Finally, the unintended load path between the two steel form systems unzipped and the front two-thirds of the girder broke free and 200 tonnes of steel and concrete plunged over 60 metres to the valley floor. The sequence of events is well illustrated in the earlier paper by Tilby (1981). Fortunately, the rear section of launching girder from the south abutment out to Pier 6 stayed in-tact and continued supporting the four Span 6 pours. Emergency works after the collapse included completing the piercap pour and a short stub of Span 5 box before stressing the Span 6 tendons and relieving the launching girder of its load.

When construction finally recommenced in late 1979 we assembled the rear 28m of new truss under span 6 and used that support to build the cantilever out from Pier 6 to the superstructure back to the arrangement needed to start a typical span cycle. We then had the interesting challenge of having to assemble the remaining front 98m of the new truss, launch it forward till its rear end would just clear the new cantilever and then hydraulically lower it 12m before rolling it back and splicing it onto the 28m section parked under Span 6.

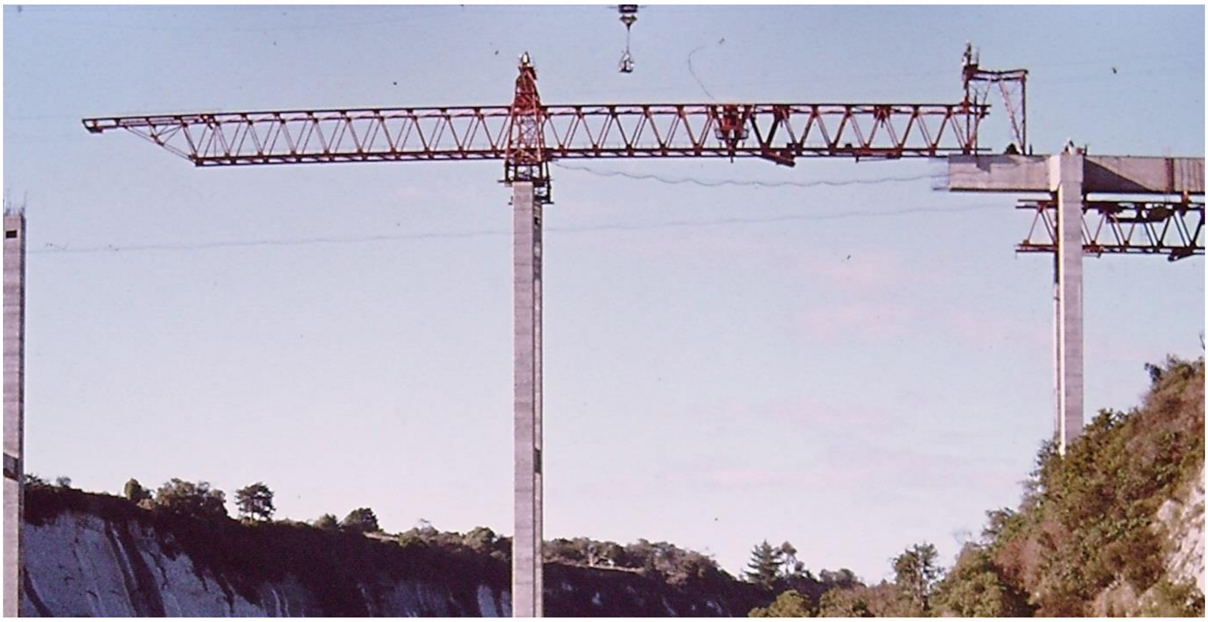

Figure 3 – Front 98m of New LG almost ready for lowering 12m and splicing to 28m tail.

ACCESS AND SITE ESTABLISHMENT

Figure 4 – the 3tonne capacity Blondin cable way was supported on towers spaced at 450m and delivered workers, equipment and concrete to any working level over a 14m plan width.

Figure 5 – Background from left to right: workshop, cableway tower slewed to left, 1m³ CIFA batching plant, canteen, and office and control room for flying fox. Visible in the foreground are the temporary towers, transverse truss and hydraulic jacks at Pier 5 during assembly LG.

Figure 6 Concrete Pump beneath 1m³ CIFA Batching Plant. Concrete was pumped up to 320m along top of bridge.

Figure 7
Slick line along top of bridge with batching plant and flying fox tower in background

DEFLECTION CONTROL

The concrete box girder section was almost twice as stiff as the launching girder that supported it and it had very little bending strength until all five pours had been completed and the entire span post-tensioned. And even though the launching girder main span was nearly 9m shorter than the typical bridge span, it would have deflected about 100mm under the loading from the first four segment pours in each span cycle. So several features were incorporated into the temporary works and the construction procedures to eliminate the risk of unacceptable cracking occurring in the superstructure during the last four pours in each typical span cycle. Some of these features were included in the original temporary works design, but others were added after the more detailed assessments that followed the collapse.

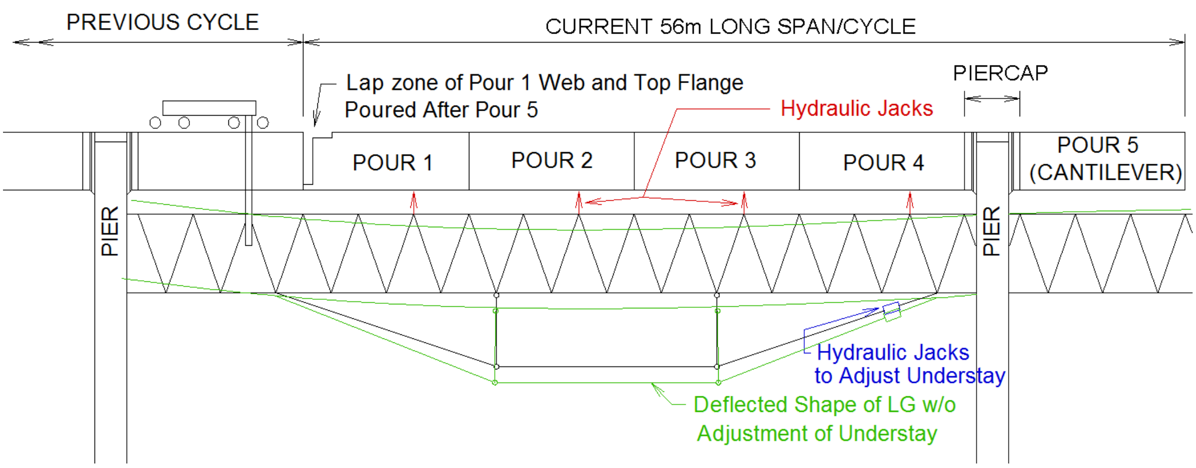

Figure 8 – Typical Span, Deflection Mitigation Measures

As the 80 tonne steel form was advanced to a new pour location, hydraulic jacks were activated to 'prop' completed segments off the launching girder. To reduce the risk of displacement due to hydraulic equipment failure, steel 'cans' and shim plates were incorporated around each jack (see Figure 11 and red arrows in Figure 8 above). Jack loads were equalized and shims adjusted prior to each new pour. Larger lock-ring jacks were used at one end of the understay system (see Figure 10 and blue lines in Figure 8 above) and the load in these jacks was also adjusted prior to each pour to help compensate for the increasing elastic deflection that would otherwise be occurring with each successive pour. Our analysis of the combined permanent works and temporary works for all the construction stages in a typical span showed that all these measures would still not be enough to prevent unacceptably high stresses being introduced at the construction joint between the first pour and the front end of the cantilever segment from the previous cycle – mainly during the placement of the Pour 2 concrete – if Segment 1 was poured full depth against the end of the previous cantilever – as had originally been planned. The downward movement of the launching girder during Pour 2 placement would have overstressed the reinforcing in the top flange of the concrete box at that construction joint. To eliminate this overstress situation we opted to form a structural gap in the webs and top flange at this joint. The length of the gap matched the lap length of the bars in those zones. While this effectively created a pinned joint at bottom flange level, we determined that the bottom flange alone had adequate strength to resist the shear load demands throughout the remaining stages of each span's cycle. The structural gap section was poured during the final stages of the combined Pour 5 (cantilever) and Piercap pour, so that we had a continuous box and adequate strength (31MPa minimum) when the span was post-tensioned.

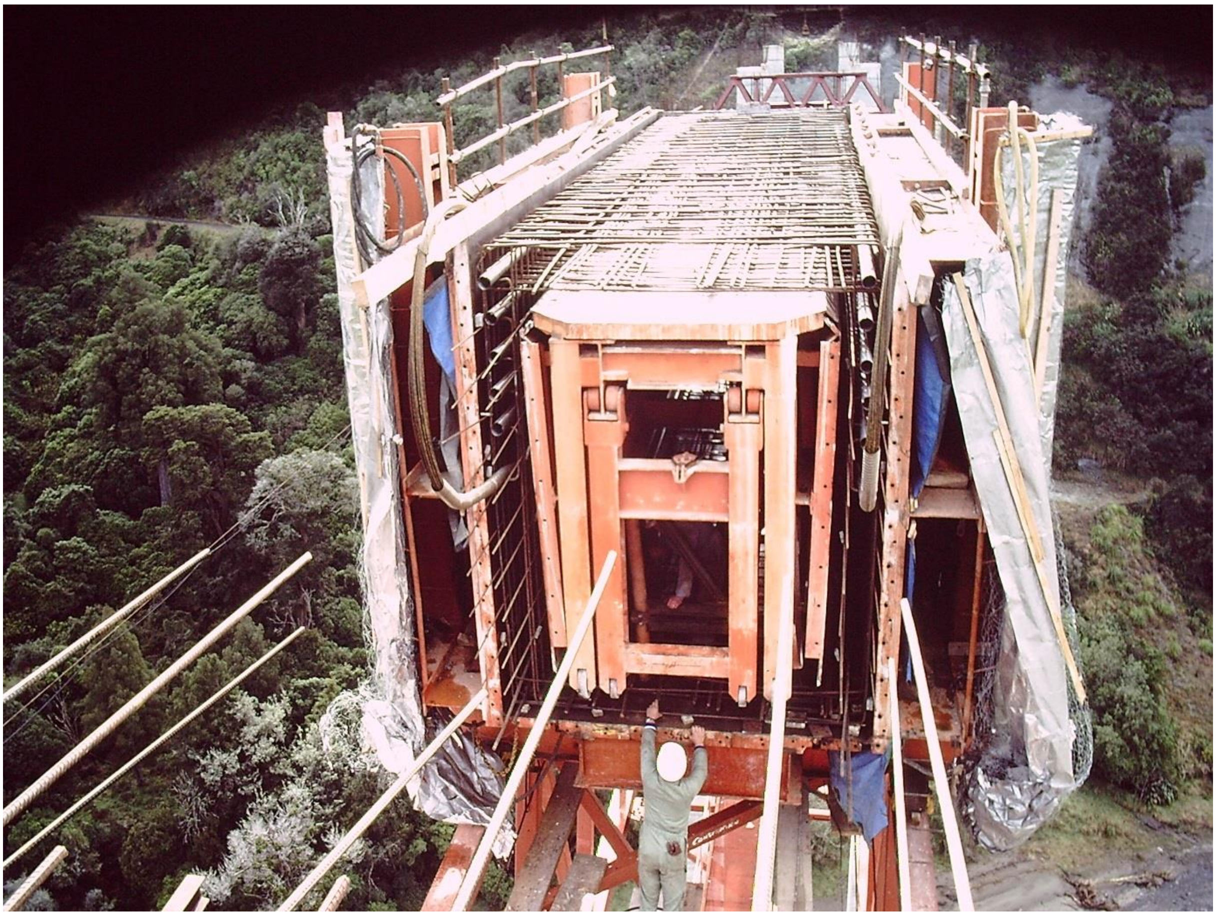

Figure 9 – 80tonne steel form – temporarily over-advanced, cleaned, oiled and rebar installed for cantilever segment while waiting for piercap formwork and rebar to installed concurrently. Rolled back to join piercap form prior to combined pour.

Figure 10 – Lock-ring jack for adjusting load in Macalloy bar understay system.

Figure 11 – Packing system and shims around hydraulic jacks supporting each segment until completed span post-tensioned.

The deflection control steps discussed above were very effective at eliminating all the elastic deflections that the launching girder would have experienced without these measures and adjustments, but we had to make sure that we didn't 'break the back' of the new span during the post-tensioning phase. We couldn't simply stress all the tendons and then release the built up load in the launching girder. The locked in stresses from that 100mm of potential DOWNWARD elastic deflection had to be progressively released. We decided it would be too difficult and risky to attempt this multistage operation by adjusting the load and shims at the jacks under each pour. Instead we released the load in the understay lock-ring jacks in three stages during the post-tensioning process.

TEMPORARY FIXITY OF PIERS

Although the base isolated, stepping piers were designed to have hinged joints at their base in the final condition, they had to be self-supporting as very tall, slender cantilevers until their tops were connected by the concrete box superstructure. This fixity was achieved by installing a pair of multi-strand tendons inside each corner of each hollow leg to tie the pier leg to the pile cap. These tendons were released after the superstructure was completed.

TEMPERATURE EFFECTS

While the pier legs were free-standing, having the sun shine on one face all morning was enough to cause the top of those legs deflect approximately 75mm away from a neutral position. By evening, with the sun shining on the other face, differential temperature effects were great enough to cause the pier top to deflect back by 150mm or 75mm towards the other side of a neutral position. The longitudinal bracing system we developed easily locked the pier we were about to incorporate with the superstructure. It incorporated a ring frame around each pier leg with spaces to drop in timber wedges when the pier had migrated to the 'correct' location. The actual forces required to restrain the piers longitudinally were not large. Longitudinal temperature effects were little more difficult to accommodate. The superstructure was stressed back to the south abutment throughout the construction of the superstructure, so any thermal expansion and contraction occurred relative to that end of the bridge. By the time the superstructure reached the northern abutment we knew we were likely to experience longitudinal thermal movements from the combined superstructure and launching girder of up to 50mm over a time frame of one calendar month. We made allowance for this by keeping the launching girder locked to the pier legs nearest the rear and introduced roller bearings under the bottom chord nodes of the launching girder at their ground supports in a short trench cut into the cliff adjacent to the north abutment (see Figure 12). The top chords had to extend approximately 10m longer - right up the abutment – with no web members to support them. At top chord nodes where the web members had been removed we introduced sliding bearings (see Figure 13).

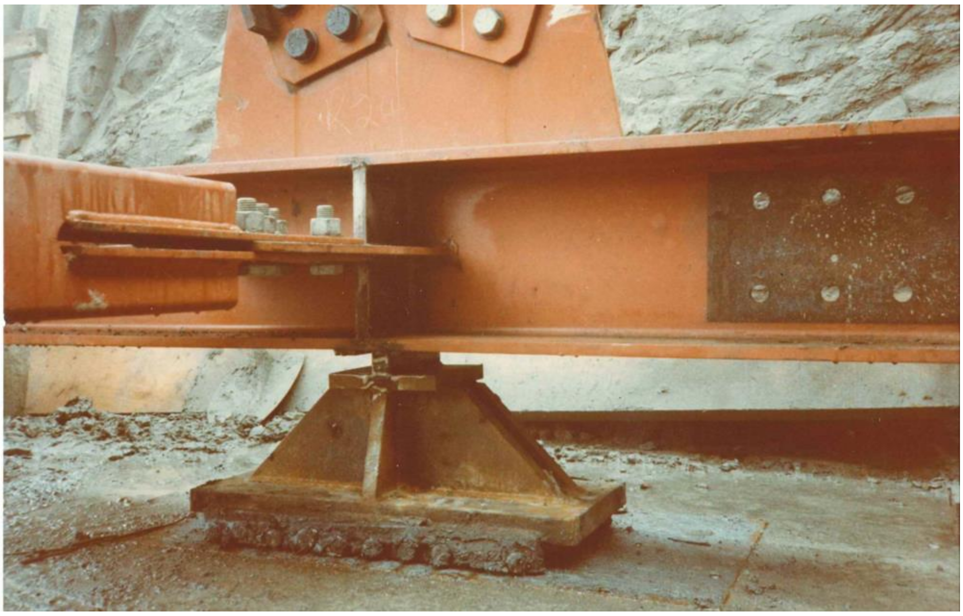

Figure 12 Needle roller bearings under bottom chord nodes

Figure 13 Sliding bearings under top chord nodes near North Abutment

ACKNOWLEDGEMENTS

John Illingsworth, Peter North, Ken Patterson-Kane, Jeff Bilkey and Paul Phillips all contributed to the design, analysis or review associated with construction engineering for the South Rangitikei Rail Bridge after the 1975 collapse.

REFERENCES

Cormack, G. L. (1988) “The Design and Construction of the Major Bridges on the Mangaweka Rail Deviation” ”, IPENZ Transactions Vol 15, No 1/CE

Tilby, C. (1981) “South Rangitikei Railway Bridge Construction”, IPENZ Transactions Vol 8, No 2/CE