Seismic retrofit design of State Highway 8 Clutha River Bridge, Alexandra

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ABSTRACT: The State Highway 8 Clutha River Bridge in Alexandra is an important link within Alexandra on the main road from Central Otago to Dunedin. The bridge is a 158 m long 3-span through-truss, continuous over two in-stream reinforced concrete piers. This paper presents a summary of the geotechnical investigations, seismic analysis and design of seismic retrofit completed by Opus. This project was commissioned as part of the New Zealand Transport Agency's (NZTA) programme for seismic improvement of State Highway bridges. As a result of the detailed seismic assessment, retrofit of the bridge was recommended and designed. Retrofits included installation of rock anchors, upgrade from roller bearings to elastomeric bearings, strengthening of shear keys, tying of the concrete deck to the transoms, upgrade of linkage bars, and change from a rigid backwall to knock-off block at the expansion joint. Retrofit work was designed for annual exceedance probability (AEP) ground shaking of 1 in 1000 years, with provision to prevent collapse for 1 in 2500 AEP event. The expected cost of the retrofit is a reasonably small proportion (9%) of the replacement cost of the bridge, which indicates the seismic retrofit is worthwhile and should proceed.

1 INTRODUCTION

This paper presents a summary of the geotechnical investigations, seismic analysis, and design of seismic retrofit for the SH8 Clutha River Bridge, Alexandra (CRB). This project was commissioned as part of the New Zealand Transport Agency's (NZTA) programme for seismic improvement of State Highway bridges. Opus was commissioned to investigate the complex CRB.

1.1 Importance, Location, and Traffic Information

The CRB is an important state highway asset being on a main route from Central Otago to Dunedin. It is also an important link within Alexandra and for several local roads that access it as a service town. The estimated Annual Average Daily Traffic (AADT) across the bridge is 6500 vehicles per day. If the bridge were to be closed, additional travel time for a detour is in the order of 30 mins.

Being a through-truss it is a visually impressive bridge and a well-known landmark. Modifications that significantly change the appearance would likely come under public scrutiny.

Figure 1: General view of the Clutha River Bridge

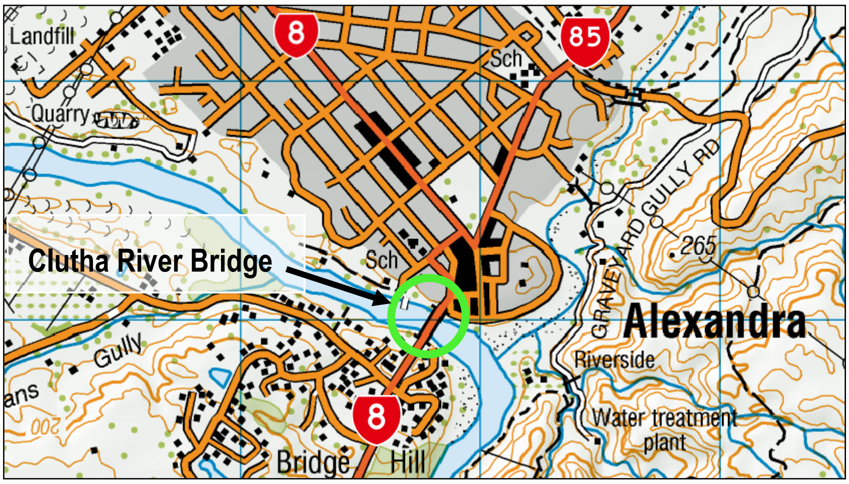

Figure 2: Bridge Location

1.2 Structural Description

Opened to traffic in 1958, the 158 m long steel bridge is a 3-span through-truss bridge (36 m - 86 m - 36 m) continuous over two in-stream reinforced concrete piers. The bridge has a bowstring-arch main-span and two Pratt trusses for the land-spans. Truss chords are typically combination members made up of smaller rolled steel sections (Channel Sections and I-Sections) with welded battens and riveted backing plates. At the piers and southern abutment, the superstructure is supported on steel rocker bearings, while at the northern abutment it is supported on pinned pedestals with linkage bars providing longitudinal restraint. The reinforced concrete deck is discontinuous with six joints at 28.6m centres along its length. The bridge has a 1 in 25 gradient up to the southern abutment.

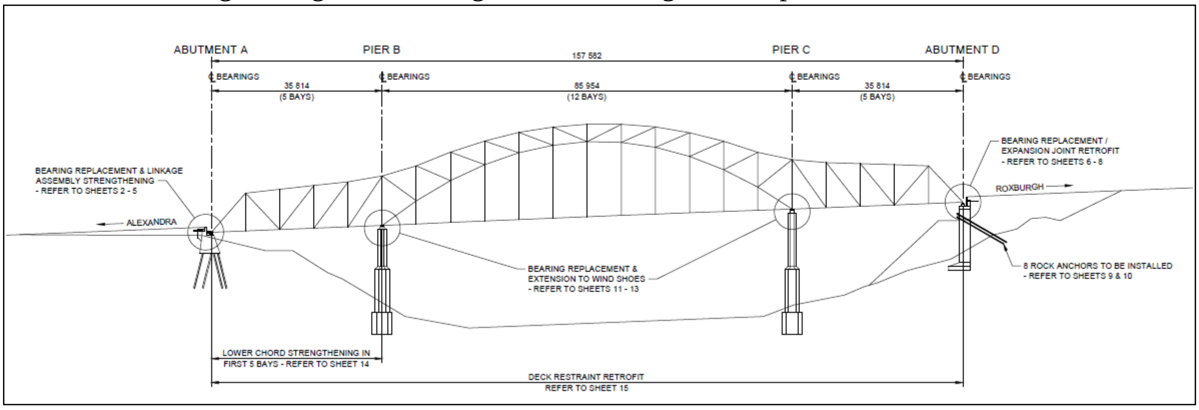

Figure 3: Bridge Elevation

The bridge carries two lanes of traffic and a separated footpath attached to the upstream side of the deck. The overall width is 10.0 m and the carriageway width is 7.3 m. Sewerage and water-supply pipes, telephone and electrical wiring are carried beneath the footpath.

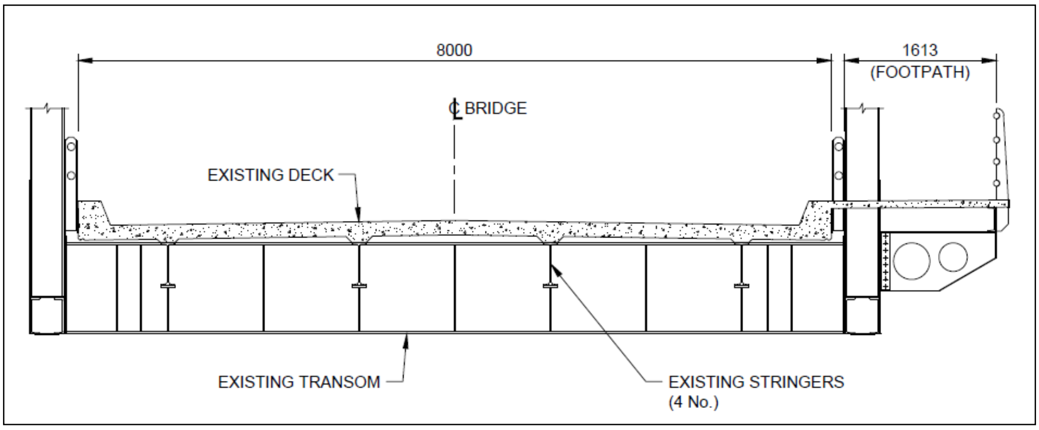

Figure 4: Bridge Typical Section

2 GEOLOGY, SEISMICITY AND GEOTECHNICAL ASSESSMENT

2.1 Geology

The geology of this area comprises generally of schist bedrock on the southern side of the river and alluvial gravels over schist bedrock on the northern side.

The bridge piers and southern abutment of the bridge are founded on schist rock. The southern abutment has an approach fill embankment formed over bedrock and dense silty gravel. The piles of the northern abutment penetrate through alluvial sands and gravels and are founded either in dense alluvial gravels or supported on schist.

2.2 Site Investigations

Site investigations included a desk-top study of the site geology, site walkover inspection by Opus geotechnical engineer, geological mapping by Opus engineering geologist, two logged boreholes, and a topographical survey.

2.3 Seismicity

The seismic hazard in terms of ground shaking, using the NZS 1170.5:2004 Hazard Factor, Z, was 0.21. The site was categorised as a Class C subsoil site, based on thickness of soils in the boreholes. Site hazard peak ground acceleration for 1/1000 AEP was PGA = 0.36g.

2.4 Slope Stability

As part of our geotechnical investigations, an Opus engineering geologist carried out geological mapping of the exposed rock face at the eastern and western sides of the south abutment, where there is no alluvium or fill blanket over the rock face. Our rock mapping at the southern abutment indicated that the schist foliation dips into the slope and therefore is unlikely to affect the stability of the southern abutment slope. The joints dip out of the slope, but their dip is between 65 and 80 degrees, whereas the slope angle at the southern abutment is 38 degrees to the horizontal. The joints were not expected to have an adverse effect on the stability at this abutment due to the difference in angle.

At the north abutment, the slope angle is shallower than that at the south abutment, and the soil-rock interface is at 18 m depth and is expected to be approximately horizontal. Therefore, potential failure surfaces are not expected to penetrate into the rock.

Based on borehole test data, the site was assessed to have low potential for liquefaction.

Our stability analysis of the abutment slopes indicated they have adequate stability under static conditions. Under seismic conditions, the abutment slopes are likely to experience some permanent displacement. Calculated critical accelerations are 0.17 g for the South abutment and 0.2 g for the North abutment. The assessed permanent displacements of abutment slopes for three different seismic events are summarised in Table 1 below.

Table 1: Assessed Permanent Displacement of Abutment Slopes

Annual Probability of Exceedance for Considered Seismic Event	Approximate assessed post-earthquake displacement of south abutment (mm)	Approximate assessed post-earthquake displacement of north abutment (mm)
1/500	40	10
1/1000	100	60
1/2500	260	170

3 SEISMIC ASSESSMENT

3.1 Assessment Criteria

The assessment criteria were to meet the performance requirements of the NZTA Bridge Manual. This meant assessing the performance of the bridge following 1/1000 AEP and 1/2500 AEP seismic events. A forced-based equivalent static linear analysis was considered sufficiently accurate to meet these assessment criteria. Probable material strengths were used in the assessment. These are based on the design strengths, factored-up according to the NZSEE AISPBE (2006) recommendations. The strength reduction factors follow the recommendations of the same document. Software used in the seismic assessment included the following:

- SAP2000 finite element model of the transoms under longitudinal and transverse loading.
- Microstran to model the performance of the superstructure under transverse seismic loads. This included assessment of the displacement of the foundations using Winkler springs, replaced with equivalent loads when ultimate soil passive resistance limit is reached.
- Response 2000 (modified compression field theory) for flexural analysis of concrete sections.
- Excel spreadsheets for analysis of concrete sections and steel members.

3.2 Longitudinal Seismic Analysis

The fundamental longitudinal period was found to be between 0.1 s and 0.5 s (dependent upon the assumed stiffness of the north abutment and longitudinal compression/tension of the bridge). Therefore the plateau value of 2.36 was chosen for the Spectral Shape Factor $C_h(T)$. This gave a Seismic Coefficient of 0.52 g for 1/1000 AEP. The key elements and member actions on the path of longitudinal inertia loads from the deck to foundations are as follows:

From the RC deck slab, longitudinal loading is carried by shear-connectors into the stringers, which in-turn load the transom web at mid height by shear through the stringer bolts and cleats. From the transom, the load is carried by web bending into the horizontal K-braces and the truss longitudinal bottom chords. The truss lower longitudinals carry axial load to the northern abutment, passing through the kingpin pedestal assembly into the linkage bars and abutment backwall. The raked piles then work mainly in axial tension/compression, with some bending, to deliver the loads into the rock and soils of the north abutment.

Deficiencies were found in the transoms, the lower chord longitudinals, the north abutment (anchorage assembly, linkage bars, holding down bolts, and backwall) and the south abutment (wall bending and displacement). Rocking of the piers was considered acceptable.

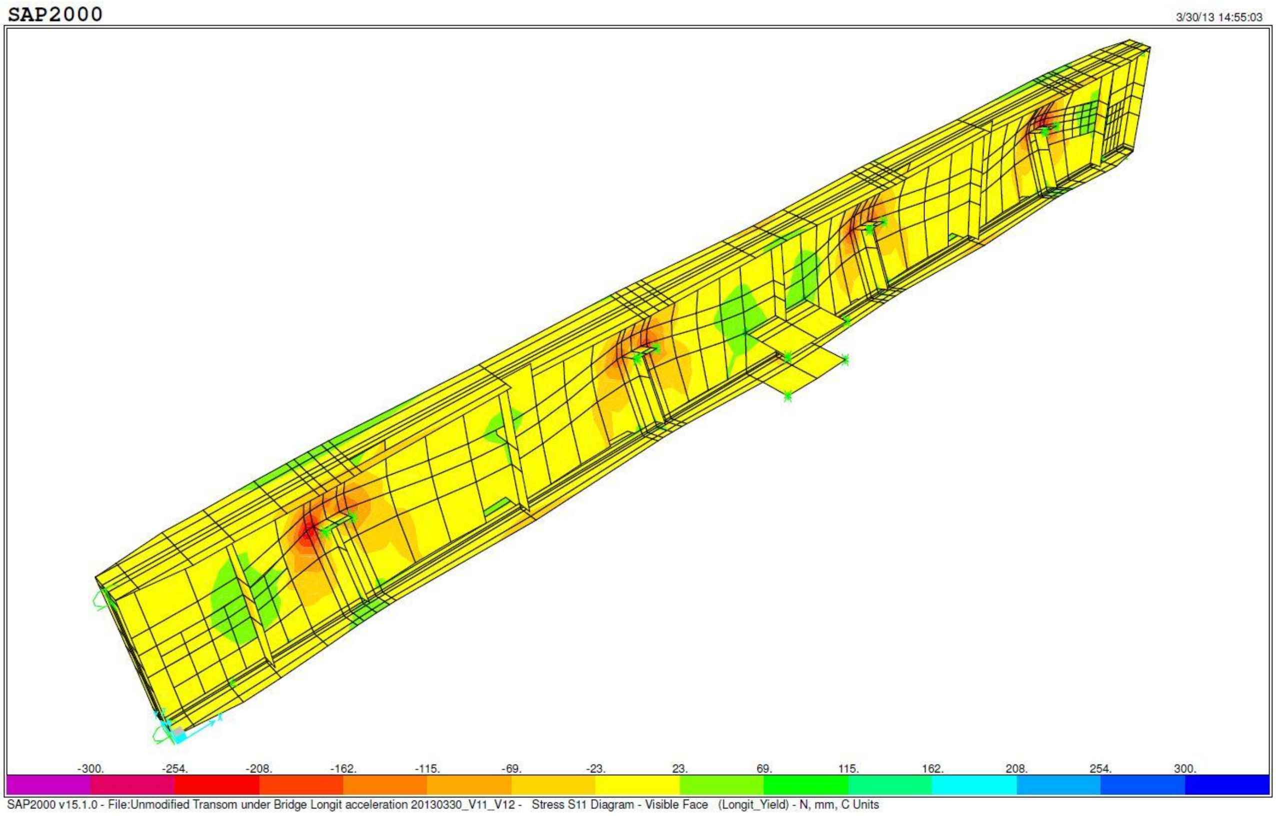

Figure 5: Transom Longitudinal Analysis Model

3.3 Transverse Seismic Analysis

The transverse period was approximately 0.8 s based on the dynamic analysis of the superstructure in our Microstran model. The Spectral Shape Factor $C_h(T)$, for a period of 0.8 s and Class C soils is 1.41. This gave a Seismic Coefficient of 0.31 g for 1/1000 AEP. The key elements and member actions on the path of transverse inertia loads from the deck to the foundations are as follows:

The RC deck slab undergoes in-plane bending and shear, transferring the load through shear connectors into the stringers. The stringers carry the load by web flexure (out-of-plane) to cleats positioned at mid-height of the transom webs. While the stringers remain elastic, there is greater shear transfer where the deck is discontinuous i.e. to the transoms at the movement joints. Inelastic deformation of the stringers is predicted to distribute the load evenly to all transoms. The transoms carry the load axially from the stringer cleats to the K-braced frame of the lower plan truss. This truss then carries the load to transoms at the piers and abutments transferring the load axially to the steel windshoes and into the concrete shear keys at the supports. Piers and abutments work in flexure and shear to deliver the load to the soils and rock of the foundations.

Deficiencies were found with out-of-plane bending of the stringers, the rocker bearings and also at the pier shear keys. Details of the rocker bearings are discussed in 3.6 below.

3.4 Base Isolation Analysis Results

Base-isolated bearings were considered as a retrofit option to reduce seismic demand on the bridge. Due to the significant wind loading, the lead plug size was maximised to the limit of the bearing dimension i.e. $d_{plug} < H/1.25$. Analysis results indicated that the effective seismic coefficient for 1/1000 AEP may be reduced from 0.31g to 0.13g by using base isolation.

A preliminary check on serviceability wind loading at 1/25 AEP found the predicted 355 kN applied to each pier bearing would exceed the characteristic shear strength, Q_d of the selected lead plugs. This indicated further deflection analysis was required. The effective stiffness of the combined lead-plug and bearing system was estimated. Based on this stiffness and 1/25 AEP wind loading, deflection of the pier bearings would be 80 mm. This slightly exceeded the 50% shear deflection limit for 600 x 600 x 201 bearings. Robinson Seismic Ltd provided results from tests on a similar sized bearing with a 175 mm dia. plug. The test system used a cycle rate of 200 s/loop in an effort to imitate the effect of wind loading. The results of the test indicate that displacements of about 150 mm could be expected which clearly exceeds the 50% shear deflection limit. At this pier displacement, some deck joints can be expected to open by 15 mm. The significant difference between calculated and tested maximum displacements may be due to the relatively slow cycle rate used in the test run.

Based on these results no further investigation into a lead-rubber bearing option was recommended.

3.5 Seismic Hydrodynamic Effects

Additional loading from seismic hydrodynamic effects was considered for the piers and found to have a significant influence on the pier longitudinal demands. The method prescribed by Kiyokawa et al (1983) was used to calculate hydrodynamic pressures and added mass from oscillation of a rectangular cross-section rigid body. It takes into account the aspect ratio and variation of pressure with depth. For a 12.2 m wide pier with a wetted depth of 6 m we found an additional pressure on the pier of 2200 kN for a 1/1000 AEP earthquake. This peak force was added to the bearing shear force and to the plateau value inertia force of the pier which was found to be sufficient to initiate rocking at 1/1000 AEP seismic demand. Predicted pier-top displacement of 100 mm was considered to be acceptable.

3.6 Rocker Bearing Analysis

Poor seismic performance of rocker (roller) bearings similar to those on the CRB is well recognised from events such as Loma Prieta and Kobe. FHWA Report 06-032 “Seismic Retrofitting Manual for Highway Structures” Figure 4-3 identifies very similar bearings as seismically vulnerable. The allowable longitudinal movement of the bearings at CRB is limited to about +/- 90 mm by a) the geometry of the pintle plates b) the base plate grooves and c) by the segmental shape of the rollers.

The bearings at Abutment D are permanently displaced about 60 mm due to movement from earth pressure on the abutment backwall and from deck temperature movement. Predicted displacement of the retrofitted Abutment D under 1/1000 AEP longitudinal ground shaking is 30 mm. Therefore, the existing bearing would at least require re-positioning of the base plate and re-shaping of the pintle plate to accommodate combined temperature and seismic demands.

Longitudinal displacement of 200 mm due to combined 1/2500 AEP event and temperature movements and/or out-of-phase movement at the north abutment would be sufficient for the rollers to fall onto their flat sides, dropping the deck at least 180 mm (7”).

Transverse loading at the piers and Abutment D is notionally intended to be via the windshoes. These have a close fit gap of +/-3 mm. However, a smaller gap, +/- 1.6 mm, is apparent between the roller and the guide strip fixed to the bearing plates and rocker plates. The interface stress between roller and guide strip is likely to be critical and the roller is likely to dislodge under moderate seismic loading.

Considering vertical movement, the bearing stability relies on self-weight only as a means of holding the superstructure down onto the bearings. Vertical acceleration or substructure displacement that causes lift-off may dislodge the roller guide brackets allowing rollers to push over onto their flat sides.

Accounting for the above shortcomings, replacement with elastomeric bearings was recommended.

4 RETROFIT DESIGN

4.1 Retrofit Criteria

In accordance with the NZTA Bridge Manual, the retrofit work was designed, for ground shaking of 1 in 1000 years AEP, with provision to prevent collapse for a 1 in 2500 AEP event. Costing for the retrofit design was required to come under 30% of the estimated re-build cost to gain approval to proceed. Constructability design was influenced by the requirement for the bridge to remain open to at least one lane of traffic at all times.

Steel materials to be used in the retrofit measures were required to satisfy the requirements of NZS3404 in relation to the possibility of brittle fracture of steel components under low temperature conditions. The design service temperatures of -15^{O}C for non-yielding elements and -25^{O}C for yielding elements were adopted. Our specification requires respective steel elements to meet the standard Charpy impact resistance of 27J at these temperatures.

4.2 Linkage Bar Upgrade

Existing linkage bars are proposed to be upgraded to 32 mm Macalloy 650 stainless steel bars with the shank turned down to 27 mm diameter for a predicted yield at 1/1000 AEP, see Figure 5. Ultimate tensile strength of the bars is approximately equal to the predicted demand from 1/2500 AEP. Available elongation of 40 mm is expected to provide sufficient ductility. The new linkage bars are designed to have threaded couplers cast into the backwall for simple replacement if bars are stretched after a severe earthquake.

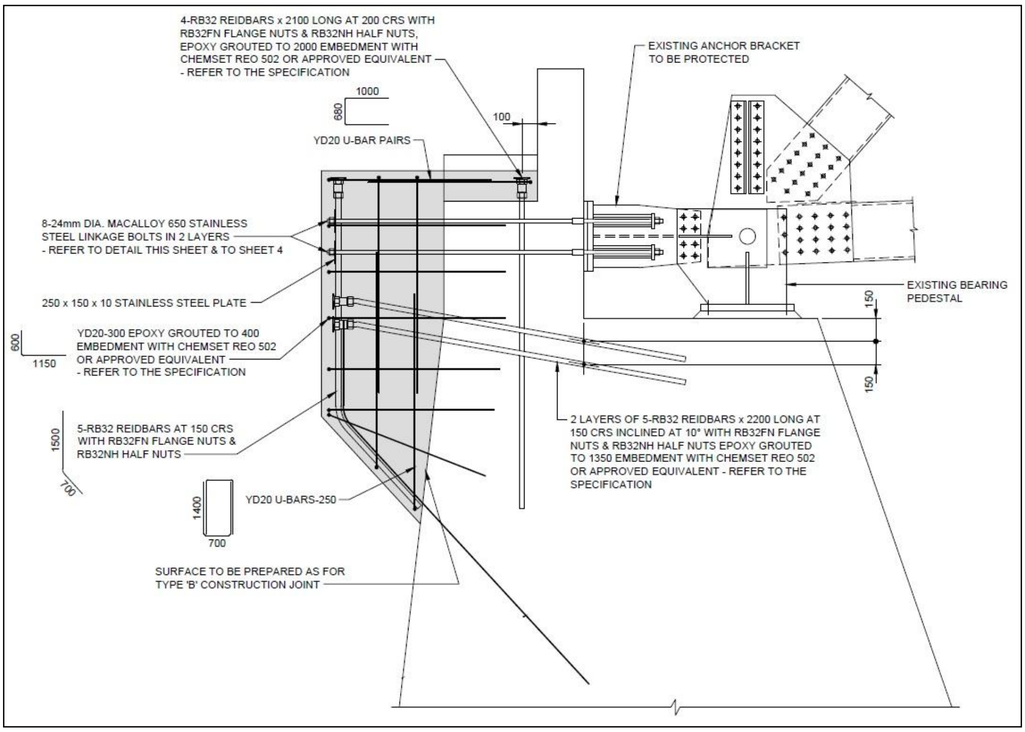

Figure 6: North Abutment Linkage Bars Upgrade and Concrete Strengthening

4.3 Shear Block Connection

New shear block connections are proposed to provide a direct load path from the deck to the transoms. This is intended to avoid the existing load path that can cause out-of-plane bending of the stringer webs. The new steel blocks will be short steel bars, 50 x 50 x 120 high, and sheathed with a light rubber tube of 2 mm wall thickness. The tube sheath allows relative movement between the non-composite deck and the transom under serviceability live loads and temperature demands. Without this sheath, the restraint of relative movement between the transom and concrete deck may fatigue the welded connection or crush the concrete at the interface.

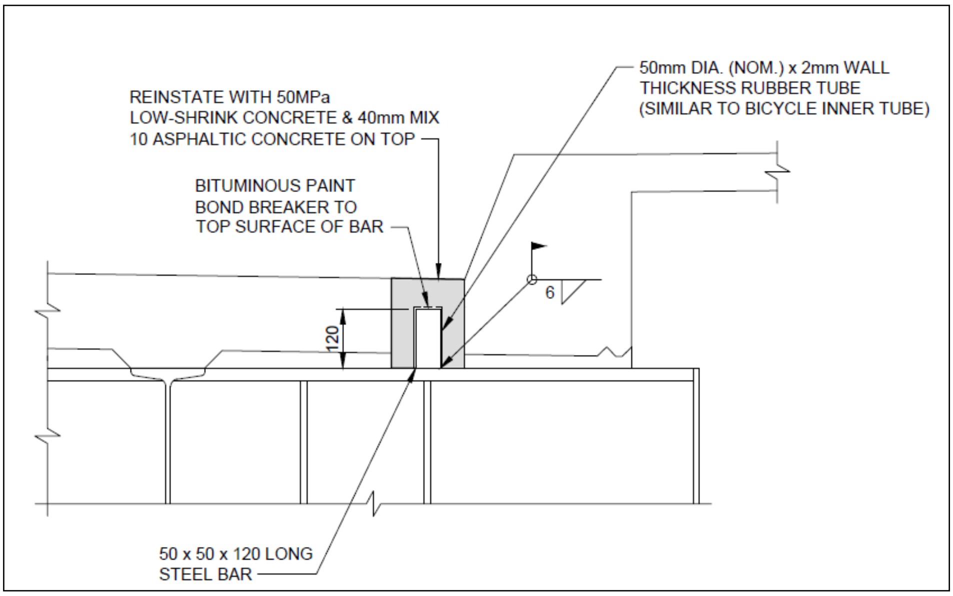

Figure 7: Shear Block Connection

4.4 Bearing Replacement

For installation of new bearings, the holding-down bolts (31.8 mm dia.) from the existing bearing plates will be left as starter bars for the mortar bearing pads. This overcomes the likely problem of drilling dowel holes when access is tightly constrained by the transom and longitudinal chords. At jacking positions, various types of strengthening are required including new welded bearing stiffeners, packer plates, and upgrade of transom connections. At pier jacking positions twin jacks, heavy bearers, and concrete confinement plates are required. Refer Figure 7 below (existing “rockers” shown in dashed lines). Installation of temporary jacking points for replacement of the bearings is likely to be one of the most expensive elements of the project. Provision of jacking points at original design stage would have been advantageous and is recommended for any new design.

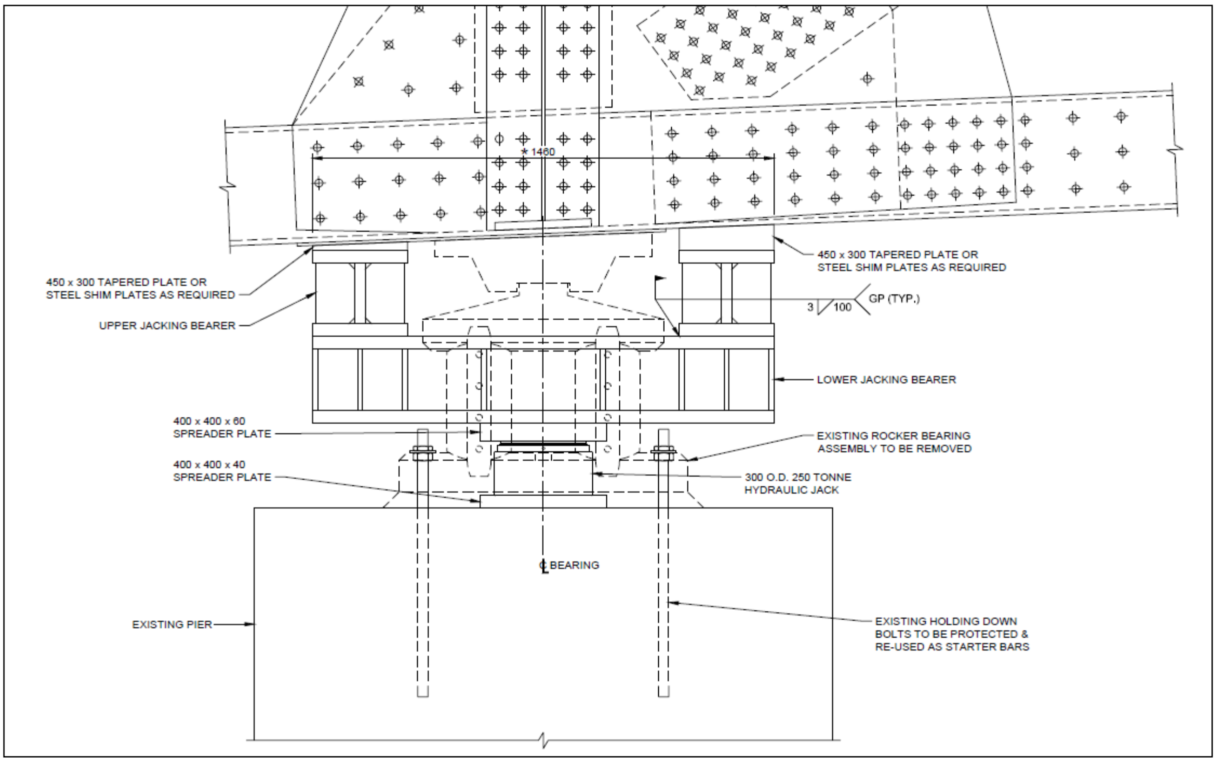

Figure 8: Pier Bearing Replacement Design

4.5 Expansion Joint and Knock-off Block

As noted in 3.6 above, the south abutment has moved towards the bridge deck by some 60 mm. To allow for this and for temperature movement, we proposed to shorten the deck and stringer stubs by about 70 mm and install a new expansion joint with a ±40 mm movement allowance for temperature. The abutment backwall is to be retrofitted with a knock-off block which safely displaces for movements greater than 40 and up to 125 mm. The max displacement allows for a combination of temperature and 1/1000 AEP movement of the bridge deck and abutment.

4.6 Rock Anchors

For stabilisation of the south abutment with rock anchors, we proposed 8 No. 38 mm diameter Freyssinet+HSA bars, double-corrosion-protected, drilled and grouted into the schist. The rock anchor will be tested to 85% of bar UTS, and then locked off at 30% of UTS. The nominal prestress of 30% of UTS is prescribed to minimise the abutment displacement required to engage the anchors bars. The anchor bars are expected to reach their yield strength at 1/1000 AEP with a predicted extension of 30 mm. As the relaxation properties of the bar may be significant, the anchor heads are designed to allow for re-stressing 1 year after construction. A bond-length of 3 m is required within competent schist, based on 2 MPa bond strength. The unbonded length will be 10 m based on the plan location and depth-to-schist at this abutment .

5 PERFORMANCE OF BRIDGE WITH PROPOSED RETROFITS

During 1/1000 AEP longitudinal ground shaking, the following may be expected:

- North abutment displacement of 40 mm.
- Pier-top displacements of 50 mm with the pier beginning to rock from the base of the pier-wall.
- Formation of a plastic hinge near the base of the buried south abutment columns.
- Predicted displacement of 30 mm at the top of the South abutment
- The vertical stiffener at midspan of the transom reaches its plastic section capacity so it may have some permanent deformation requiring repair.
- Initiation of yielding in the linkage bars.

The resulting damage should be minor allowing the bridge to remain in service following 1/1000 AEP ground shaking.

For stronger ground shaking up to 1/2500 AEP we expect significant damage to several components but a loss of support is not likely considering a) the large displacement capacity of retrofitted elastomeric bearings, b) adequate span-support overlap provided at both abutments c) likely confinement provided by the north and south abutments, and d) additional damping arising from yielding of steel members to reduce the demand.

For the transverse retrofitted case at 1/1000 AEP, the superstructure and foundations are expected to remain elastic. For stronger ground shaking, up to 1/2500 AEP, the following may be expected:

- Onset of yielding in the north abutment steel H-piles from 1/1700 AEP.
- The concrete deck slab may yield in in-plane bending. The deck has poor ductility capacity because the concrete cracking moment is greater than the moment due to yielding of the reinforcing. Therefore a low cycle fatigue failure may be possible.

Loss of support is unlikely as the pile hinging should not be sufficient to form a collapse mechanism and the cracking of the concrete deck should be limited by support from the stringers and transoms.

6 COST ESTIMATES

The estimated replacement cost is $9,400,000 for this complex medium span bridge. The expected cost of the proposed retrofit is $815,000. The expected retrofit cost is, therefore, about 9% of the bridge replacement cost. This gives a strong indication that retrofit is a cost effective option to secure the bridge against earthquakes.

7 CONCLUSIONS

This paper presents a summary of the analysis, design and costing to retrofit the CRB to meet the design level earthquake of 1/1000 AEP, with provision to prevent collapse at 1/2500 AEP.

Innovative retrofits included re-configuration of load-paths to bypass weak elements for a cost-effective solution. Installation of temporary jacking points for replacement of the bearings is likely to be one of the most expensive elements of the project. Provision of jacking points at original design stage would have been advantageous and is recommended for any new design.

Detailed analysis accounted for seismic hydrodynamic effects on the piers.

The analysis of a base isolation option was found to be not feasible due to serviceability forces arising from wind loading.

8 ACKNOWLEDGMENTS

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9 REFERENCES

Federal Highway Administration, Office of Infrastructure Research and Development, January 2006, Figure 4-3, *FHWA Report 06-032 Seismic Retrofitting Manual for Highway Structures Part 1 Bridges*.

Kiyokawa T., and Kobayashi H., March 1983, Characteristics of Added Mass Due to Oscillation of Submerged Cylinder Having a Plane Symmetrical Cross-Section, *Research Bulletin No. 2* Shimizu Institute of Technology.

NZSEE Study Group on Earthquake Risk Buildings, June 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, New Zealand Society for Earthquake Engineering (incl Cor. No.1)

New Zealand Transport Agency, 2003, *Bridge Manual Second Edition including amendments June 2004, September 4004, and July 2005,* New Zealand Transport Agency.