Seismic assessment of the Clarence River bridge

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ABSTRACT: The Clarence River Bridge is a balanced cantilever, post-tensioned box girder bridge located on State Highway 1, approximately 30 kilometres north of Kaikoura in the South Island. The bridge is approximately 305m long and comprises 6 spans. Design of the bridge was completed around 1968 by the Ministry of Works. The bridge was constructed in 1969. Consequent to a nationwide seismic screening project, the bridge was assessed in detail in 2003 and 2004 to determine its seismic performance. This paper presents the findings of the assessments and the proposed seismic retrofitting solutions to protect the bridge.

1 INTRODUCTION

For some years now Transit New Zealand has engaged in a programme of screening its State Highway bridges for seismic resistance and assessing for retrofit those bridges identified as being the highest route priority. Linkages retrofitting has almost been completed on the highest priority bridges. Detailed assessments have been undertaken on the 50 bridges of highest priority for assessment. The Clarence River Bridge is one of the 50 bridges. As part of the nationwide programme of assessment work (Transit New Zealand, 2000a), the seismic resistance of SH 1 Clarence River Bridge was assessed in detail to:

- Determine the seismic performance and identify vulnerabilities to earthquake loading as well as their probabilities of occurrence.

- Identify possible bridge retrofit options necessary to enable the bridge to satisfy the seismic performance requirements specified in the Transit New Zealand Bridge Manual (Transit New Zealand, 2000).

- Establish preliminary cost estimates for bridge retrofit and associated benefit/cost ratio.

An initial assessment (Opus, 2003) found that the response of the bridge structure was sensitive to liquefaction. Consequently, geotechnical investigations were undertaken in 2004 and the liquefaction potential of the bridge site was studied to assess the response of the bridge with greater certainty. This paper summarises the findings of the assessments.

2 DESCRIPTION OF THE BRIDGE

The Clarence River Bridge is located at RP 118/0.00 on State Highway 1, approximately 30 kilometres north of Kaikoura. Photograph 1 shows the bridge and Figure 1 shows the location of the bridge.The post-tensioned box girder bridge was constructed using the cast-in-situ, balanced cantilever method. The depth of the superstructure varies from 3.2m at pier supports to 1.1m at mid-span. The superstructure is integral with the reinforced concrete piers and expansion joints are present at mid-span of each main span. It carries a single-2 carriageway road over the estuary of the Clarence River, and comprises four 61m main spans and two 30.5m end spans. The bridge is on a horizontally curved alignment with a radius of approximately 582m; and a vertical grade of 1.1%. Design of the bridge was completed around 1968 by the Ministry of Works.
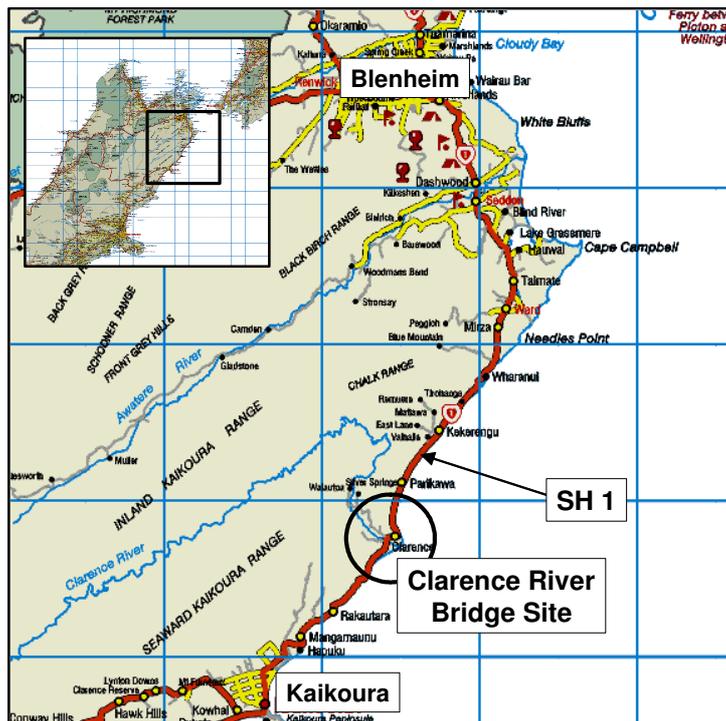
Photograph 1: Clarence River Bridge

Figure 1: Clarence River Bridge site location

Piers, which are of an elongated octagonal shape, are each founded on a pair of 2.4m diameter cylinders with belled bases. The cylinders comprise hollow sections, in-filled with gravel, with in-situ concrete plugs at the top and bottom. Piers vary in height from 6m to 9m in height whilst the cylinders are 12m in length.

Elastomeric bearings support the end spans at the abutments. Tie-down bolts are provided to the end spans, at the abutment, in accordance with standard practice at the time the bridge was designed. Abutments are of spill-through form, founded on steel H-piles through the embankment fill with three front piles raked at 1H:8V whilst the rear two piles are vertical. Figure 2 shows the general arrangement of the bridge.

3 TOPOGRAPHY, GEOLOGY AND SEISMICITY OF THE BRIDGE SITE

3.1 Topography

The topography of the site comprises generally flat river terraces that form the Clarence River valley. The Clarence River flows in an incised channel that runs from the Kaikoura mountain ranges towards the west of the Clarence River Bridge and then continues 2.5km to the east coast of the South Island.

The river has steep sided terraces on either side. The road is located on a river terrace about 10m above the riverbed. The meandering braided river flows on a flat gravel bed with vegetated flood plains on either side.
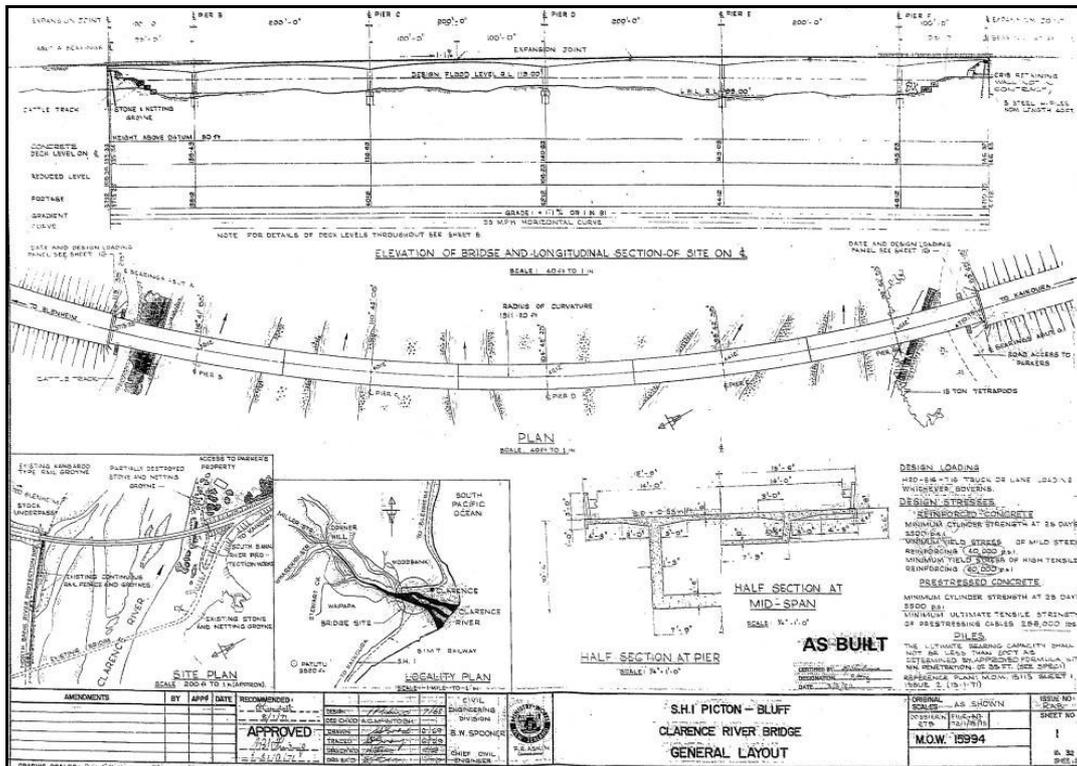

Figure 2: Clarence River Bridge general arrangement

3.2 Geology

The New Zealand Geological Map for the area (Department of Scientific and Industrial Research, 1962) indicates the geology of the site to be alluvium, sand silt and gravel of Holocene age. The available information on the ground conditions from the 1969 and the 2004 site investigations confirms this geology at the site. The terrace on either side of the river is underlain by the Parikawa Formation, a sandy mudstone and silt.

On either side of the river further back are well-rounded conglomerate deposits with large sub-angular limestone blocks and there is a limestone quarry on the south side of the river further upstream.

3.3 Seismicity

There are several major active faults in close proximity to the Clarence River Bridge, they are listed below and described by the Institute of Geological and Nuclear Sciences (Stirling, et.al., 2000).

Hope Fault (Conway-Offshore) - Offshore, 5km southeast from the bridge. It is capable of rupturing with a horizontal displacement of 4.5m has a slip rate of 23mm per year and a magnitude 7.5 earthquake at a return period of 200 years.

Jordon Fault - Located 10km northwest from the site. It is capable of a 7.1 magnitude earthquake with a 3m horizontal displacement at a recurrence interval of 1,200 years.

Kekerengu Fault - Situated 10km west from the Clarence River Bridge. It is capable of a 7.2 magnitude earthquake with a 5.5m horizontal displacement at a recurrence interval of 730 years.

Clarence Fault NE - Fault located about 25km northwest from the bridge. It has a slip rate of 4.7mm per year, and is capable of a 7.7 magnitude earthquake with a 7m horizontal displacement at a recurrence interval of 1,500 years.

Hundalee Fault - Located 40km southwest from the bridge. It is capable of a 7.0 magnitude

earthquake with a 1.5m horizontal displacement at a recurrence interval of 2,000 years.

3.4 Earthquake loading spectra

Earthquake loading for the bridge was determined from site specific spectra that were developed and recommended by the Institute of Geological & Nuclear Sciences (McVerry, et.al., 2003). These spectra as well as the maximum credible earthquake (MCE) spectrum are depicted in Figure 3.
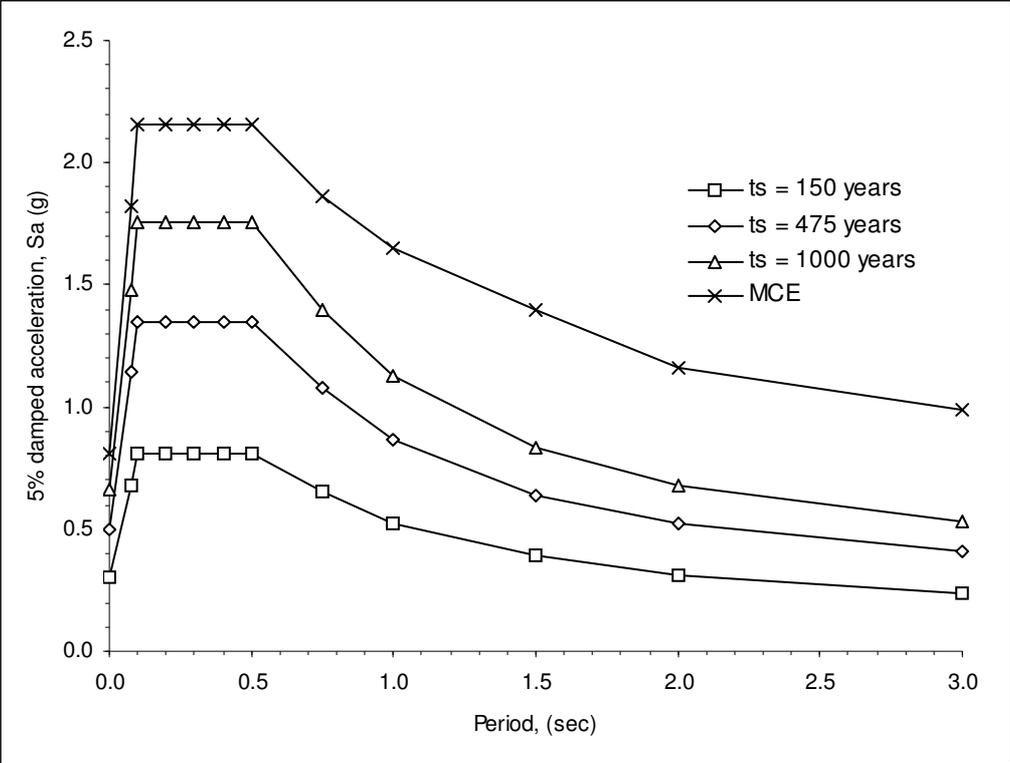
Figure 3: Site specific spectra for the Clarence River Bridge (after IGNS)

A detailed examination of the site specific spectra suggested that the spectra developed by IGNS for hazards with a return period of 150, 475 and 1000 years respectively were consistent with each other via a scaling factor. The scalability of the site specific spectra inferred that analytical results (within the linear range) of the structure based on one spectrum could therefore be factored accordingly to derive the response to a spectrum with a different return period.

4 GEOTECHNICAL ASSESSMENT

4.1 1969 Investigations

Given the proximity of the site to a number of active faults, the bridge could potentially experience high levels of seismic shaking during its future life. The ground water table at the site is high. The Ministry of Works carried out investigations at the bridge pier locations in 1969. As part of the 1969 investigations, 152mm diameter casings were driven at pier locations. Ground conditions encountered and the blows per foot of the casing penetration into the ground were recorded on the Ministry of Works casing logs. Although the logs indicated that the concrete cylinders supporting the bridge piers were founded in alluvial sandy gravels, material descriptions given in the logs are not detailed nor were Standard Penetration tests (SPT) and grading tests undertaken. In the absence of the grading test data, SPT data, correlations between SPT and casing blowcounts, and information on the efficiency of the hammer used to drive the casing, it was difficult to assess the density and liquefaction potential of the in-situ materials at the bridge site.

4.2 Initial Assessment

As a result, the initial seismic assessment of the Clarence River Bridge was constrained by the uncertainty with respect to the liquefaction potential of the site materials. To address this uncertainty, initially, the seismic assessment was undertaken for two different scenarios.

In the first scenario it was assumed that the top 6.5m thick layer of sandy gravels had low potential for liquefaction and would not liquefy in an earthquake. In the second scenario it was assumed that the top 6.5m thick layer of sandy gravels had high potential for liquefaction and would liquefy in an earthquake. The sandy gravels below 6.5m depth were assumed to have low potential for liquefaction for both scenarios as the 1969 casing blowcounts below 6.5m depth were generally higher compared to those in the top 6.5m thick layer.

Geotechnical parameters for the detailed modelling and seismic analysis of the bridge for both scenarios were derived from the available geotechnical information and scenario-specific assumptions with respect to the properties of site materials. The geotechnical parameters used to describe static and seismic response of the site soils included material densities, angles of internal friction, cohesion, moduli of elasticity and constants of horizontal subgrade reaction. These are discussed in detail in the Opus seismic assessment report (Opus, 2003).

The initial seismic assessment undertaken for both scenarios concluded that the bridge performance is very sensitive to liquefaction and recommended geotechnical investigations to address the uncertainty with respect to the site liquefaction potential and refine the conclusions of the initial assessment. The client adopted this recommendation and the geotechnical investigations were scoped and undertaken in 2004.

4.3 2004 Investigations and Conclusions

The 2004 geotechnical investigations were specifically scoped to assess the liquefaction potential for the bridge site. Opus carried out borehole investigations at the bridge site in September - October 2004. The 2004 investigations comprised drilling of two investigation boreholes to 20 m depth with associated SPT and sampling.

The 2004 borehole data indicated that the bridge piles are founded in dense to very dense sandy gravels. The ground water table level measured in the boreholes corresponded well to the river level observed at the time of the investigations. The site potential for liquefaction was assessed based on the 2004 borehole data, using computer program LiquefyPro (CivilTech Corporation, 1998) and found to be low. Given the low liquefaction potential of the site soils, the assessment scenario with liquefaction of the top 6.5 m thick soil layer (the second scenario) was not considered to be a credible scenario for the assessment of the bridge seismic behaviour. Therefore, the first scenario (low liquefaction potential of the site soils) was adopted for the assessment of the seismic performance of the bridge.

5 MODELLING AND ANALYSIS OF THE EXISTING BRIDGE

5.1 General assessment methodology

The response spectrum method was employed to evaluate the bridge response to seismic excitation. A structural performance factor (Sp) of 0.67, being appropriate for the site and with respect to nominal ductility, was applied. Although it was recognised that some nominal ductility (generally $\mu=1.25$) will be available in the reinforced concrete structure, a structural ductility factor of $\mu=1$ was adopted for simplicity. Higher ductility factors were not assumed in cognizance of the detailing, which was not aimed at providing ductility in structural elements at the time the bridge was designed and constructed.

Longitudinal and transverse earthquake loading responses were combined in accordance with the Bridge Manual methodology to account for the simultaneous occurrence of earthquake shaking in two orthogonal horizontal directions. The respective earthquake loading cases were then combined (per Bridge Manual requirements) to determine the worst case elastic demand on the respective bridge elements.

Assessment of the bridge was generally undertaken in two stages. In the first stage, the existing bridge was assessed to study its dynamic characteristics (which would indicate its response to seismic excitation), identify the critical elements and to determine the hierarchy of element failure until a collapse mechanism forms. This was achieved by iterative analyses and adjusting the properties of critical elements based on its secant stiffness.

In the second stage, the bridge was assessed on the basis that critical elements that were identified in the first stage have been strengthened, and the bridge upgraded as appropriate, to meet the demand as specified by current design standards.

5.2 Structural modelling

The bridge was modelled as a linear three-dimensional space frame with SAP2000 (Computer & Structures Inc., 2003) to capture the effects of the curved alignment. Modelling of the bridge structure was based primarily on the recommendations given by Priestley et. al. (1996). Piers and pilecaps were modelled using cracked section properties whereas the post-tensioned superstructure and cylinders were initially assumed to be uncracked. Rigid offsets were used to simulate joints.

Abutments were modelled as horizontal springs whose stiffnesses were derived in consideration of force-displacement relationships as the bridge pushes in or pulls away from the abutment/approach embankment. Tension and compression abutment models were adopted to investigate the influence of the abutment/approach embankment reaction to the overall bridge response.

Stiffness of the mid-span hinge hanger and shock absorber were modelled with representative axial, flexural and torsional stiffnesses. Interaction between the cylinders and subgrade was modelled with Winkler springs. The respective soil spring stiffness was derived based on an assessment of the available geotechnical investigation information. Initially, the loss of lateral support to the foundations due to soil liquefaction was also investigated.

The mass of the bridge was calculated based on the cross sectional areas of the respective member and material density. A constant modal damping of 5% was adopted; and the Complete Quadratic Combination (CQC) method was selected to combine the modal results. Small amplitudes and displacements were assumed in the modal analysis.

6 DYNAMIC CHARACTERISTICS OF THE BRIDGE STRUCTURE

A description of the first ten modes of vibration as observed in the bridge model are summarised in Table 1. The cumulative mass participation ratios for the first 30 modes are presented in Figure 4.

Mode		1	2	3	4	5	6	7	8	9	10
Period (sec)		1.359	1.304	1.283	1.232	1.194	1.165	1.047	0.972	0.957	0.752
Modal Mass Participation (%)	UX	12.0	0.6	0.0	0.0	93.4	23.8	4.2	0.9	3.9	37.6
	UY	88.0	99.4	100.0	100.0	6.6	76.2	95.8	20.6	95.9	11.5
	UZ	0.01	0.0	0.0	0.0	0.0	0.0	0.1	78.5	0.2	50.9
	RX	21.6	57.1	85.3	45.6	49.4	78.1	30.4	42.0	31.8	0.0
	RY	0.1	2.1	3.3	0.0	50.0	1.8	0.1	0.0	0.1	100.0
	RZ	78.3	40.8	11.4	54.4	0.6	20.1	69.5	58.0	68.0	0.0

Table 1: Dynamic characteristics of the bridge (first 10 modes)

Coupling between longitudinal and transverse modes of vibration was not significant due to the relatively shallow curvature of the alignment. However, the presence of the hinges influenced the coupling of modes (or lack thereof), particularly the transverse, superstructure twisting and substructure twisting.

It was found that the hinges played a significant role in the dynamic characteristics of the bridge. The hinges introduce a discontinuity in the stiffness at the mid-span locations. As a result, the bridge responds, at the lower frequencies of vibration, as five tied cantilevered "T" frames. These modes of vibration dominated the first four modes. Load transfer across the hinges resulted in the twisting of substructure as the form of distortion that required the least energy. Mass participation in the longitudinal and transverse directions due to these modes of vibration was low as expected.

Longitudinal translational modes were concentrated in the fifth and sixth modes, with up to 84% of the mass activated. Of the two modes, the fifth mode is of particular importance as it activates 73.0% of the total mass in the longitudinal direction. Transverse modes of vibration were mainly spread over the first nine modes. This spread was primarily due to the presence of the hinges and the low torsional stiffness of the substructure as a whole.
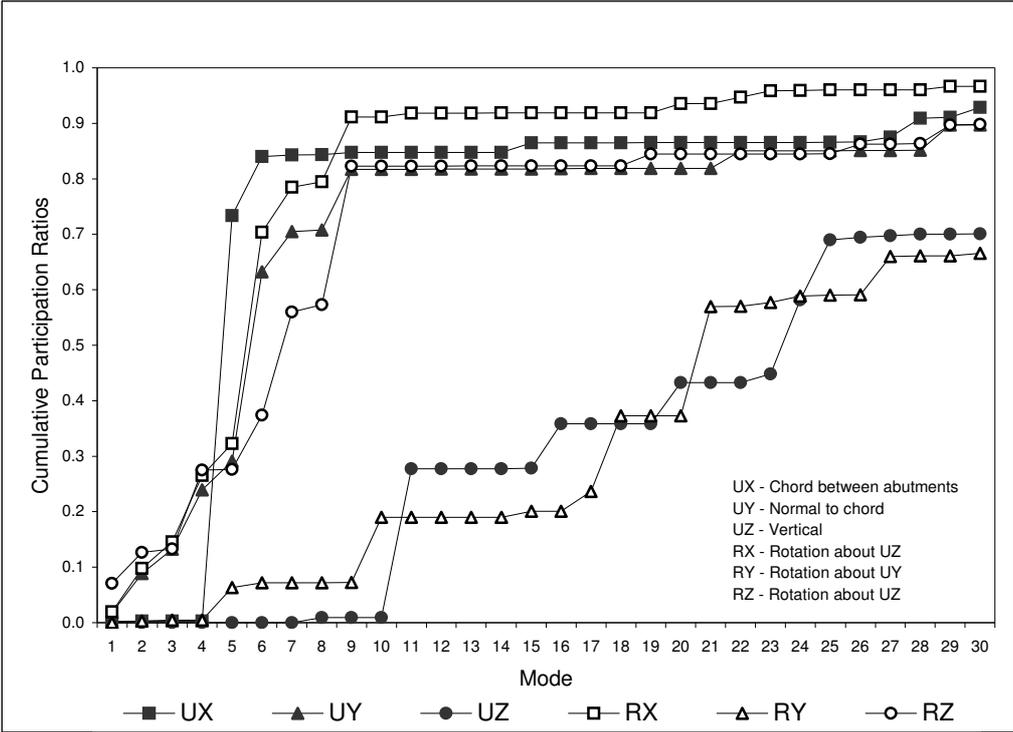

Figure 4: Clarence River Bridge cumulative mass participation ratios

7 VULNERABLE ELEMENTS OF THE BRIDGE AND CONSEQUENCES OF DAMAGE

Vulnerable elements and their associated critical return period of earthquake damage are summarised in Table 2.

Depending on the location and depth of the slip surface, deep seated slope failures in the approach fill may displace the abutments permanently. Abutment piles may also be damaged in such an event. However, deep seated slope failure is not expected to cause (directly or indirectly) structural distress to the bridge superstructure which was designed and constructed as a balanced cantilever.

Damage to the abutment walls and backfill would cause only temporary disruption to bridge use until the damage is repaired and the carriageway is reinstated. The damage is inconsequential to the structural integrity of the bridge and abutment.

Distress in the mid-span hinge would not render the bridge unusable. However, any subsequent damage in the pilecaps or piers would result in structural instability that will lead to bridge collapse. Damage to the cylinder in the form of flexural hinging whilst under tension would also lead to structural instability. Damage to the pier-superstructure joint diaphragms will lead to permanent deflections in the superstructure, putting the bridge out of service until the damage is repaired.

Bridge Element		Critical Return Period of Damage	Mode of Damage
Approach fill		100 years	Deep seated slope failure
Abutments	Backwall and Wingwall (including backfill)	<10 years	Flexural / Passive soil failure behind Abutment
	Sidewall	35 years	Flexural
Mid-Span Hinges	Shock Absorbers	15 years	Bolted connection /shock absorber piston yields
	Vertical Hangers	190 years	Yielding of bolted connection to bridge
Cylinders[1]		300 years	Tension / Flexural
Pilecaps[1]		300 years	Torsional
Piers[1]		970 years	Torsional
Pier-Superstructure Joint[1]		1100 years	Shear

[1] After yielding of Abutment and Mid-Span Hinges

Table 2: Summary of Vulnerabilities

8 RETROFIT

Possible strengthening of vulnerable bridge elements to mitigate damage are tabulated in Table 3.

Element	Strengthening Works
Approach fill	Regrade slope for higher degree of stability as desired.
Abutments	Do nothing. Retrofit to prevent damage to the walls is not recommended in view that backfill deformation cannot be prevented. However, the principal basis for not strengthening the walls is that early failure of the walls will protect the abutment piles from damage.
Mid-Span Hinges	Reconstruct the hinge as a reinforced concrete hinge (replacing shock absorbers and hangers). New hinge to be capacity protected.
Cylinders	Construct additional piles. New piles to be capacity protected.
Pilecaps	Construct new pilecap in conjunction with new piles.
Piers	Increase shear capacity with more reinforcement and steel jacketing. Steel jacket also to increase ductility in flexure and torsion.
Pier-Superstructure Joint	Increase capacity by building up and improving confinement with concrete and reinforcement.

Table 3: Element strengthening

In order to comply with Bridge Manual standards, all vulnerable elements would require strengthening (full retrofit) thus altering the bridge structural configuration. As an alternative and although this will not meet Bridge Manual standards, partial retrofit (strengthening some but not all elements) could be implemented at a lower cost to progressively reduce the risk of crippling seismic damage. Bridge retrofit options (comprising combinations of element strengthening) that were considered were:

Option	Description	Estimated Cost
A	Retrofit fills around abutments only (partial retrofit)	$150,000
B	Retrofit mid-span hinges only (partial retrofit, risk of crippling damage to structure reduced)	$300,000
C	Option B + Retrofit cylinders, pilecaps, piers and pier-superstructure joints (full retrofit, structure fully protected)	$5.5million
D	Option A + Option C (full retrofit, structure and fill around abutments fully protected)	$5.65million

Table 4: Bridge retrofit options (combination of element strengthening)

9 RETROFIT ECONOMICS

Economic analysis showed that none of the retrofit options are economically viable in the technical sense (B/C < 1.0). Nonetheless, Transit is committed to a modest annual programme of seismic retrofitting of critical bridges on the state highway network. The process for weighing up the cost of various options for retrofitting of any particular structure against the perceived reduction in risk for each option is still being developed.

10 CONCLUSIONS

The assessment showed that the mid-span hinges and foundation cylinders of the Clarence River Bridge are vulnerable to seismic damage. Other elements vulnerable to earthquake damage are the approach fill, abutments, piers and pilecaps.

In order to upgrade the Clarence River Bridge to Bridge Manual standards, all vulnerable elements will require strengthening (full retrofit). Alternatively, partial retrofit at a lower cost comprising mid-span hinge strengthening only could be applied to reduce the risk of crippling damage although this will not bring the bridge up to current performance standards.

Whilst it is technically feasible to upgrade the Clarence River Bridge, the retrofit options have poor economics in the traditional sense. A subsequent peer review of the assessment (not previously mentioned, but part of Transit's process) recommended that the partial retrofit (mid-span hinge strengthening) to improve the resilience of the bridge against earthquake events should be undertaken.

11 ACKNOWLEDGEMENT

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