

# SEISMIC ASSESSMENT AND RETROFIT OF WAIKANAЕ AND PAKURATAHI RIVER BRIDGES

J.H.Wood<sup>1,2</sup>, H.E. Chapman<sup>2,3</sup>, G.C. Gregg<sup>3</sup> and  
D.K. Kirkcaldie<sup>4</sup>

*This paper was presented at the 2007 NZSEE Annual Conference in Palmerston North.*

## ABSTRACT

Seismic strengthening construction work has recently commenced on the Waikanae and Pakuratahi River Bridges assessed as having a high priority for retrofitting. Both bridges are on major State Highways in the Wellington region that carry high traffic volumes. The paper describes the assessment methods and strengthening details used on these two bridges.

## 1 INTRODUCTION

In addition to extensive retrofitting of the Thorndon Over-bridge and Shelly Gully bridges in Wellington and the Auckland Harbour Bridge Transit New Zealand has been undertaking a systematic assessment of the seismic security of approximately 2,500 other state highway bridges. Preliminary screening identified 335 bridges that required detailed seismic assessment and 170 bridges that needing improvement to their inter-span linkages (Chapman *et al.*, 2005). Linkage retrofitting on 89 bridges on Priority 1 routes (motorways, expressways and state highways carrying more than 4,000 vehicles per day) has been completed. Detailed assessment and priority ranking has been completed on 50 structures. Strengthening construction work has started on the Waikanae and Pakuratahi River Bridges and investigations plus retrofit design work has commenced on 10 other high priority bridges.

The seismic performance deficiencies identified on the Waikanae and Pakuratahi River Bridges are typical of those found on bridges of similar age. Design of the bridges was based on a working stress design approach with a seismic loading of 0.1 times the weight of the elements being restrained.

Retrofitting design for the two bridges was based on the requirements of the Bridge Manual (Transit NZ, 2003) for new bridges on Priority 1 routes and satisfied the main performance criterion for new bridges of preventing collapse under a 2,500 year period event, however damage to the retrofitted bridges at this level is likely to be greater than for a new bridge.

The seismic hazard at the bridge sites was determined using the provisions of NZS 1170.5: 2004, Part 5, which gives Zone Factors (Z) of 0.4 for Waikanae and 0.45 for Pakuratahi. These values are in the middle of the 0.13 to 0.6 range for Z

indicating a moderate to high seismic hazard at the sites.

## 2 WAIKANAЕ RIVER BRIDGE

### 2.1 Bridge Description

The five-span bridge was designed in 1962 and opened to traffic in 1964. It carries the two lanes of State Highway 1 across the Waikanae River, about 55 km north of Wellington City. The superstructure is a solid prestressed concrete slab continuous over the 80 m length of the bridge. Each of the four piers is supported on two 1.2 m diameter bored piles founded on rock. The abutment seating beams are founded on four steel H piles driven to rock. Details of the bridge are shown in Figure 1.

### 2.2 Foundation Soils and Slope Stability

The foundation soil consist of layers of medium dense to very dense silty sandy gravel alluvium overlying greywacke bedrock at a depth varying between 9 to 19 m. Due to the dense nature of silty sandy gravels the site has low potential for liquefaction.

The toe of the northern abutment slope has been undercut by the river causing over-steepening of the slope. Slope stability analyses indicated that outward movements of the slope of up to 1 m were likely during a 1,000 year return period seismic event. Movements of this order were expected to result in large lateral forces on the pile foundations of the closest pier to the abutment.

### 2.3 Structural Analysis

Static push-over analyses for both principal directions of the bridge were carried out. These analyses gave estimates of the response acceleration levels for the onset of damage and fail-

<sup>1</sup> John Wood Consulting, Lower Hutt, New Zealand.

<sup>2</sup> Life Member

<sup>3</sup> Transit New Zealand

<sup>4</sup> Opus International Consultants Ltd., Wellington, NZ. (Member)

ure of the main lateral load resisting components of the existing bridge. Similar analyses were repeated for the strengthened structure to assess the degree of improvement in performance.

In the longitudinal direction of the bridge the deck links the abutments and piers together so that they are all displaced the same amount. Incremental push-over analysis is straightforward for this direction with the total applied inertia load distributed in accordance with the relative stiffness of the substructure components. Behaviour of the structure under transverse direction loading is more complex with the displacement of the abutments and piers influenced by both the stiffness of the substructure components and the stiffness of the deck acting as a continuous beam spanning between the piers and abutments. Analysis was simplified by modelling the deck as a beam on an elastic foundation with the spring stiffness values for the individual abutments and piers, including the pile foundations, determined by two-dimensional frame analysis.

#### 2.4 Performance of Existing Bridge

Response acceleration versus deck displacement for loading in the longitudinal and transverse directions is shown in Figures 2 and 3 respectively. A summary of the response acceleration levels that caused damage or failure of the lateral load resisting elements is presented in Tables 1 and 2.

Analyses were simplified by assuming that after the abutment holding down bolts and the pier shear keys fail, the connections remain rigid. Essentially the computed response is for a bridge without connection defects.

The unstrengthened bridge did not meet the 2,500 year return

period performance requirement with collapse likely at a response acceleration level about 20% lower than this level. Assuming the connections between the superstructure and substructure were strengthened, the bridge might survive a 1,200 year return period event. Because of the lack of confining reinforcement and the unsatisfactory main reinforcement lap details at the base of the pier columns, at this level of loading there would be serious damage to the piers. There would also be serious cracking and spalling in the abutment seating beams that are loaded in torsion and shear by axial loads from the raked piles.

#### 2.5 Strengthening

The ductility of the piers was improved by specifying 8 mm thick elliptical shaped steel jackets extending to a height of 2 m above the base of the pier columns. The construction contractor submitted an alternative proposal for confinement using a reinforced concrete jacket with sufficient hoop reinforcement to provide the same confinement pressures as the steel jackets. This alternative was accepted and details are shown in Figure 4.

Improving the lateral resistance of the bridge by installing pairs of large diameter bored piles at each abutment was investigated. Following detailed analysis of the strengthening options it was decided that the bridge would meet the design criteria without new piles.

The shear strength of the pier cross-beams was improved by bolting steel channel sections formed from 610 mm deep Universal Beam sections across the width of the pier top as shown in Figure 5.



*Figure 1. Waikanae River Bridge*

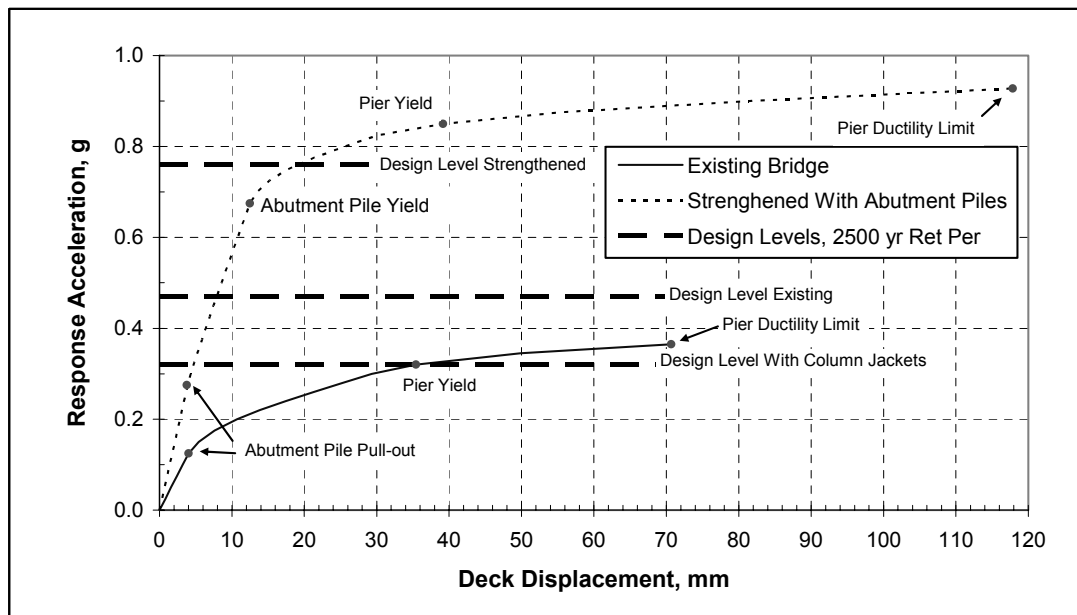


Figure 2. Performance of existing and strengthened bridge for longitudinal loading.

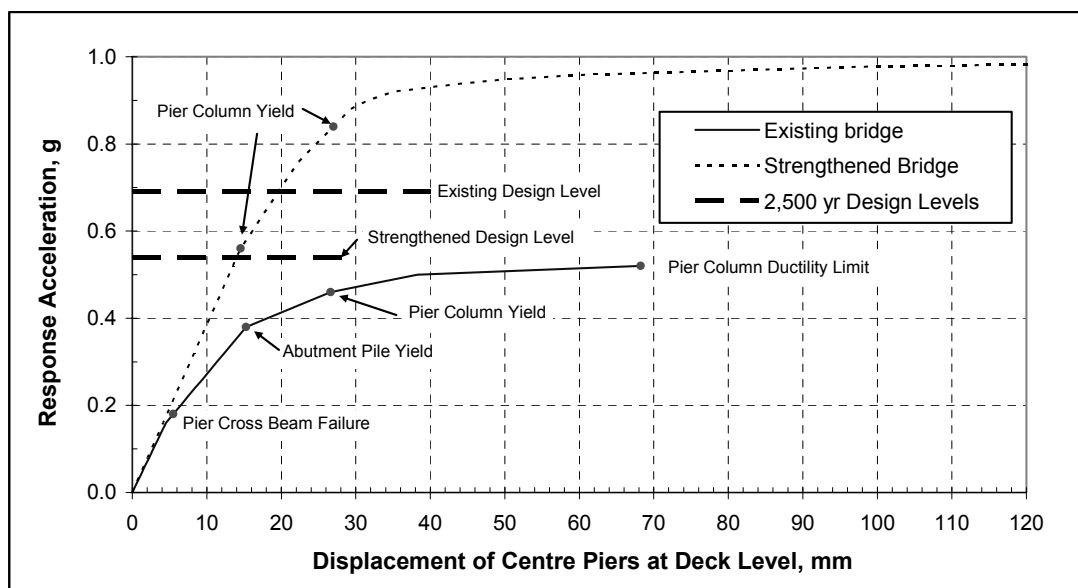


Figure 3. Performance of existing and strengthened bridge for transverse loading.

Infilling the area between the pier columns with a 200 mm thick reinforced concrete wall was also considered but the steel channel option was preferred as it strengthened the pier top against the splitting forces from the deck slab connections.

The bolted connections between the superstructure and abutments were strengthened by fixing four fabricated steel brackets to the underside of the deck at each abutment location and linking them to the abutments with 30 mm diameter medium tensile linkage rods.

The connections between the deck and the piers were strengthened using pairs of 150 mm diameter medium tensile

dowels grouted into holes cored through the deck and into the tops of the piers.

Rip-rap was designed for the toe of the northern abutment slope to improve the stability under earthquake loads and to prevent further undercutting by river scour.

**Table 1. Existing Bridge. Summary of performance: longitudinal direction.**

<b>Resp Accn, g</b>	<b>PGA, g (Ret Per), yrs</b>	<b>Deck Disp, mm</b>	<b>Description of Damage or Failure</b>
0.13	0.14 (30)	6	Pull out of two piles at each abutment loaded in tension. Not critical but deformation will cause some damage to abutments.
0.18	0.20 (60)	8	Shear failure of abutment holding down bolts followed by bridge impacting into the abutment back walls and causing their failure. Failure of bolts removes stiffening effect of abutments transferring greater loads to the piers.
0.25	0.28 (110)	20	Passive pressures against the abutment wing walls that cantilever from the abutments wall cause reinforcement yield and severe cracking in wall. These are not critical items.
0.32	0.35 (190)	35	Yield in pier reinforcement at a bar cut-off point 500 mm above base of pier. Onset of large displacements of superstructure.
0.33	0.37 (200)	50	Shear failure of pier steel tube shear keys and holding down bolts. Superstructure slides on pier tops. Possibly not very critical but serious damage likely to pier tops.
0.36	0.81 (1500)	75	Serious damage in plastic hinge regions near base of pier columns because of lack of confinement reinforcement and a lap splice in all the vertical bars at the same location.

**Table 2. Existing Bridge. Summary of performance: transverse direction.**

<b>Resp Accn, g</b>	<b>PGA, g (Ret Per), yrs</b>	<b>Deck Disp, mm</b>	<b>Description of Damage or Failure</b>
0.16	0.13 (25)	5	Combined flexure and shear failure in the pier cross beams that link the tops of the pier columns. This results in a loss of the portal frame action and a redistribution of the bending moments in the columns with the critical section becoming the bar cut-off point near the column bases. Under increasing deformations damage is likely to the deck above the cross beam failure region.
0.30	0.25 (90)	12	Shear failure of pier steel tube shear keys and holding down bolts. Superstructure slides on pier tops. Possibly not very critical but serious damage likely to pier tops.
0.30	0.25 (90)	12	Shear failure of holding down bolts at abutments. Failure of bolts removes stiffening effect of abutments transferring greater loads to the piers.
0.38	0.40 (250)	16	Yield in the steel H piles supporting the abutments with plastic hinges forming under the capping beam and at a depth of about 3 m.
0.48	0.50 (450)	35	Yield in reinforcement at a bar cut-off point 500 mm above base of pier. Onset of large displacements of superstructure.
0.52	0.81 (1500)	68	Serious damage in plastic hinge regions near base of the pier columns because of lack of confinement reinforcement and a lap splice in all the vertical bars at the same location.



*Figure 4. Reinforcement in concrete jacket.*



*Figure 5. Steel channel strengthening on pier cross-beam.*

## 2.6 Performance After Strengthening

The longitudinal and transverse displacement response curves for the strengthened bridge are shown in Figures 2 and 3 respectively. In the longitudinal direction the displacement response is similar to that of the existing bridge except that the displacement at the pier ductility limit increases to about 140 mm. For this direction a response curve is shown for the option of installing large diameter bored piles at the abutments in addition to the proposed work. These piles would significantly enhance the performance with reinforcement yield in the piers unlikely.

In the transverse direction plastic hinges form in the abutment steel H piles under the seating beam and at depth in the soil.

The retrofitted bridge is unlikely to collapse in the 2,500 year return period event but there will be significant damage to the abutment piles and seating beams and in the plastic hinge zones at the base of the piers.

## 3 PAKURATAHI RIVER BRIDGE

### 3.1 Bridge Description

The three-span bridge was designed in 1965 and was opened to traffic in 1971. It carries the two lanes of State Highway 2 across the Pakuratahi River, about 15 km north of Upper Hutt. The bridge superstructure consists of simply supported post-tensioned I beams with a reinforced concrete deck and conventional linked joints at the piers and abutments. Details of the bridge are shown in Figure 7.

The 8.5 m high tapered rectangular wall piers have a hinge joint at the base making them effectively pinned top and bottom for longitudinal response. Because the piers are 7 m wide the pinned connection to the pile cap provides significant re-

sistance to overturning in the transverse direction. The piers are supported on steel H piles driven into dense gravels.

### 3.2 Foundation Soils

The site has a depth of more than 20 m of medium dense to very dense silty sandy gravels overlying greywacke bedrock. Due to the dense nature of the silty sandy gravels the site has low potential for liquefaction.

### 3.3 Structural Analysis

The procedures used to analyse the existing bridge and the strengthening options were similar to those described for the Waikanae River Bridge.

In the longitudinal direction linkage bolts tie the abutments and piers together so that initially they all displace the same amount. The piers do not provide significant longitudinal resistance resulting in the longitudinal inertia load being distributed to the two abutments. At the west abutment the load is resisted by steel H piles, a 5.2 m long friction slab and passive soil pressures on the back-wall. The east abutment is on a spread footing with resistance provided by an 8.7 m long friction slab, passive soil pressures and friction on the base of the footing. The interaction between the various resisting components at the abutments is difficult to accurately define. In particular, the phase relationship of the inertia loads on the soil providing the weight on the friction slabs in relation to the inertia loads on the superstructure is unknown.

The span linkage bolts have rubber washers so they do not provide sufficient restraint for the deck to act as a diaphragm between the abutments under transverse loading. Analyses for both the existing and retrofitted structure under transverse loading were therefore based on a tributary load assumption with the horizontal earthquake loads on the abutments and piers divided in proportion to the span gravity loads on each. Forces in the foundations of the individual abutments and piers were determined using two-dimensional frame analysis.

### 3.4 Performance of Existing Bridge

The response acceleration versus deck displacement performance for loading in the longitudinal and transverse directions is shown in Figures 8 and 9 respectively. A summary of the response acceleration levels that cause damage or failure of the lateral load resisting elements is presented in Tables 3 and 4.

The displacements shown in Figure 8 for the existing bridge following the failure of the abutment back-walls, and for the bridge with strengthened back-walls following failure of the abutment foundations, were computed using the Newmark sliding block theory. After the failure of the back-wall and abutment foundations, resistance is derived mainly from frictional effects such as friction on the span rubber bearings, passive pressures and sliding of the friction slabs.

The longitudinal performance is improved by strengthening the abutment back-walls. However, even with this level of strengthening longitudinal displacements of up to 150 mm occur resulting in significant damage to the abutments and their foundations.

The transverse loading analysis (Figure 9 and Table 4) showed that the existing bridge did not meet the 2,500 year return

period performance requirement with collapse likely at less than the 500 year return period level.

### 3.5 Strengthening

Options of strengthening either the abutments or the piers were considered. Strengthening the piers was the option adopted. Large diameter bored piles are to be installed at either end of the existing pile caps and a ductile reinforced concrete frame pier with tapered rectangular columns and a beam section on either side of the top of the existing pier wall constructed on top of the new piles. The new columns and beams are structurally separate from the existing piers but the beams have bearing contact in both principal directions to hold the existing piers rigidly to the new frame. Details of the new concrete frames are shown in Figure 10.

The new frames were designed to carry the entire longitudinal earthquake load and the tributary transverse earthquake load from the adjacent spans with half the tributary transverse load from the two end spans carried on the abutments.

Linkage rods are to be installed at the abutments to ensure that there was no risk of the spans sliding off the abutments under the 2,500 year return period longitudinal displacements.



*Figure 7. Pakuratahi River Bridge*

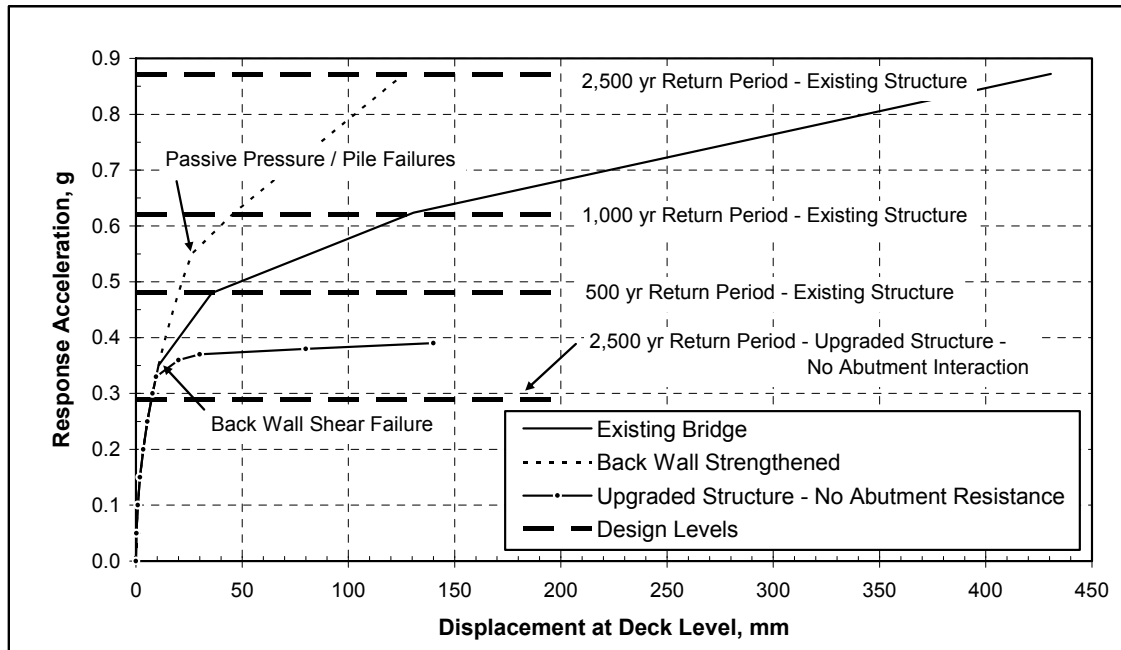


Figure 8. Performance of existing and strengthened bridge for longitudinal loading.

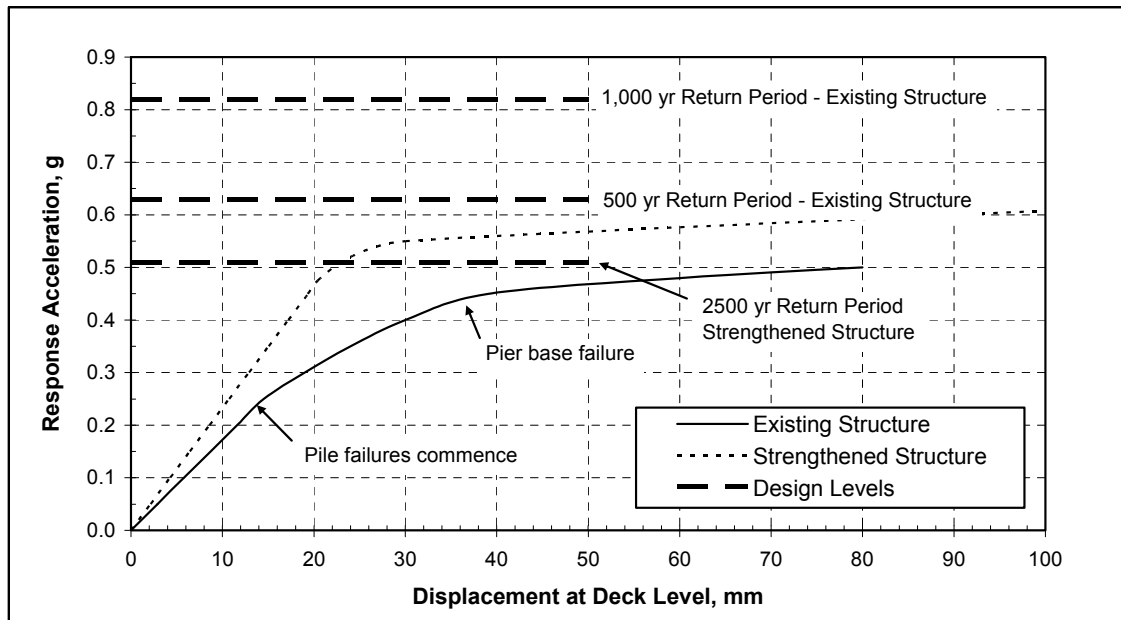


Figure 9. Performance of existing and strengthened bridge for transverse loading.

**Table 3. Existing Bridge. Summary of performance: longitudinal direction.**

Resp Accn, g	PGA, g (Ret Per), yrs	Deck Disp, mm	Description of Damage or Failure
0.34	0.43 (220)	10	Shear failure of abutment back-wall on “push” end of bridge with flexural failure on “pull” end of bridge. Resistance is then provided by passive pressure and friction on abutment bearings. Displacements controlled by damping from sliding friction.
0.55	0.69 (700)	30	On assumption that back walls are strengthened increasing response accelerations cause loads to increase on abutment foundations. Pile failures under west abutment and sliding of friction slabs at both abutments. Displacements controlled by sliding friction.
0.60	0.75 (900)	40	Passive pressures against the wing walls that cantilever from the abutments will cause the reinforcement to yield resulting in severe cracking. These are not critical items.
0.85	1.06 (2,200)	400	End spans will fall off abutment seatings leading to a high risk of collapse of the piers.

**Table 4. Existing Bridge. Summary of performance: transverse direction.**

Resp Accn, g	PGA, g (Ret Per), yrs	Deck Disp, mm	Description of Damage or Failure
0.25	0.16 (35)	15	The load capacity of the pier piles in axial compression is exceeded. Large axial deformations develop resulting in tilting of the pile cap.
0.40	0.25 (70)	50	Bond failures occur in the dowel reinforcing between the pier and pile cap (hinge joint). This allows the pier to rock on the cap followed by crushing of the pier concrete at the contact points on the edges of the pier.
0.50	0.31 (100)	100	Pull-out failures develop in the piles. With increasing loads very large deck displacements arise from both the pile and pier failures. Collapse is likely at a response acceleration of about 0.6 g.

**Figure 10. Strengthening frame under construction alongside existing pier.**



### 3.6 Performance After Strengthening

The longitudinal displacement response curve for the bridge with strengthened piers is shown in Figure 8 for the case where no resistance is assumed at the abutments. Following failure of the abutment back-walls the longitudinal load will be resisted by a complex interaction of a frictional resistance at the abutments and the ductile flexural resistance of the piers. A detailed study of the interaction was not made but it is clear that if the new piers and piles are designed to carry the full longitudinal earthquake load a satisfactory ductile response to the 2,500 year return period event will be achieved.

A deck displacement of about 130 mm is expected under the 2,500 year return period level. Although this will cause significant damage to the abutment back-walls it will not present any risk of collapse as the spans are well connected to the abutments by the new linkage bars. It is more cost effective to accept back-wall damage and the associated repair costs than to modify the abutments to accommodate the

displacement without damage. It is unlikely that this damage would close the bridge to traffic for any significant period.

The transverse displacement response curve (Figure 9) for the strengthened bridge indicates satisfactory performance at the 2,500 year return period level with moderate inelastic deformations in the plastic hinges that form in the columns of the new frame members.

## 4 DAMAGE RISK AND RETROFITTING COST

The estimated risks of loss service and collapse from earthquake damage of the existing bridges and for the strengthened bridges are summarised in Table 5. Loss of service is taken to be closure of one or more lanes for a period exceeding one day. The risk is expressed in terms of probability (percent) of loss of service or collapse within the estimated 70 year remaining life of each bridge. The cost of the retrofit work is also given in the Table and ranged from 26% to 47% of the replacement costs for the Waikanae and Pakuratahi River Bridges respectively.

**Table 5. Damage Risks and Retrofit Costs.**

Bridge Configuration	Probability of Loss of Service, %	Probability of Collapse, %	Retrofit Cost, \$ (% of Replacement Cost)
<b>Waikanae River Bridge</b>			
Existing bridge	50	10	-
Retrofitted bridge	10	0.5	\$507,000 (26%)
<b>Pakuratahi River Bridge</b>			
Existing bridge	40	25	-
Retrofitted bridge	10	0.2	\$654,000 (47%)

## 5 ACKNOWLEDGEMENT

The permission of Transit New Zealand to publish the information contained in this paper is acknowledged with thanks.

Opus International Consultants Ltd carried out the detailed assessment of the Waikanae River Bridge (Opus International Consultants, 2000) and Beca Carter Hollings and Ferner Ltd the detailed assessment of the Pakuratahi River Bridge (Beca Carter Hollings and Ferner, 2004).

## 6 REFERENCES

- Beca Carter Hollings and Ferner Ltd, (2004). "Detailed Seismic Bridge Assessment Investigation and Reporting - Pakuratahi River Bridge". Report by *Beca Carter Hollings and Ferner Ltd* for Transit NZ.
- Chapman, H.E., Lauder, M.K & Wood, J.H., (2005). "Seismic Assessment and Retrofitting of New Zealand State Highway Bridges". *Proceedings NZSEE Annual Conference*.
- Opus International Consultants Ltd, (2000). "Waikanae River Bridge Seismic Assessment". Report by *Opus International Consultants Ltd* for Transit NZ.
- Transit New Zealand, (2003). "Bridge Manual", Second Edition [SP/M/022]
- NZS 1170.5: (2004). "Structural Design Actions, Part 5: Earthquake Actions" – *Standards New Zealand*.