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PERFORMANCE OF ROAD BRIDGES DURING THE 14 NOVEMBER 2016 KAIKŌURA EARTHQUAKE

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ABSTRACT

The transport infrastructure was majorly affected by the 14th November 2016 Kaikōura Earthquake. Severe vertical and horizontal peak ground accelerations generated high inertial forces, land-slides, and liquefaction. Most of the bridges in the Hurunui, Malborough and Kaikōura districts were critical nodes to the railway and road networks. In total, 904 road bridges across those districts were affected. Two reached the life safety limit state, suffering severe damage, however, most of the affected bridges experienced only minor to moderate damage. This paper describes the structural performance of the most severely damaged bridges based on observations made from site inspections. In addition to this, several performance issues have arisen from this event and are posed in this paper, hopefully to be addressed in the near future.

INTRODUCTION

The 14th of November 2016 (NZST) Kaikōura Earthquake was special in that due to it being the summation of multiple fault ruptures over a large spatial domain, the transportation network of the entire northeast portion of the South Island was badly affected [1, 2]. The Hurunui, Kaikōura and Marlborough council districts were worst affected. In these three districts there are over 268 State Highway bridge structures (most of which are made of reinforced concrete) and 636 local road bridge structures. Table 1 below gives a breakdown of the number of bridge structures in each district.

Table 1: Number of local and State Highway bridge structures (includes culverts, stock underpasses, and rail underpasses) in the worst affected council districts.

	Managing Authority	
Council District	Local road (District Council)	State Highway (NZ Transport Agency)
Hurunui	240	105
Marlborough	348	111
Kaikōura	48	52

Shortly after the Kaikōura event, two reconnaissance groups were dispatched from the University of Canterbury to rapidly assess geotechnical and road bridge structural damage within the three districts aforementioned. One group explored the Hurunui District, whilst, the other visited the Marlborough, and a very small portion of the Kaikōura District. At the time of reconnaissance, Kaikōura Township and the whole of the Kaikōura District south of Okiwi Bay (30 km north of Kaikōura Town) was inaccessible by land and so no descriptions of damage to bridges in that area are given in this paper. A total of 28 bridges were inspected over the course of the reconnaissance: 11 in the Hurunui District, 14 in the Marlborough district and 3 in the Kaikōura District. A table of the bridges inspected is given together with a map of their locations in Figure 1 and 2. The aim of this paper is to present general observations for the performance of road bridges in the Kaikōura Earthquake.

The description of the performance of the inspected road bridges is structured into two levels in this paper. In the first level, the bridges are grouped according to the different council districts. This was chosen as a clear method to categorize bridges spatially in relation to the fault rupture sequence and implicitly capture spatial variation in ground shaking intensity. In the second level, the damage observations in each district are grouped within the different eras of design (based on reference [3]) dictated by changes in the New Zealand Transport Agency (NZTA) Bridge Manual (BM). This was chosen to attempt to capture the relationship between seismic performance and improvement in seismic design practice. The observations in this paper are focused more on structural damage, however, basic descriptions of geotechnical and utility damage are also presented to provide a complete picture of the observed damage related to bridges and their approaches. More details regarding geotechnical observations from the Kaikōura Earthquake can be found in Stringer et al. [4]. Of the bridges inspected, several displayed interesting and also complex damage patterns and for four of those bridges, simplified, damage schematics are provided to show the distribution of damage and indicate their response during the earthquake.

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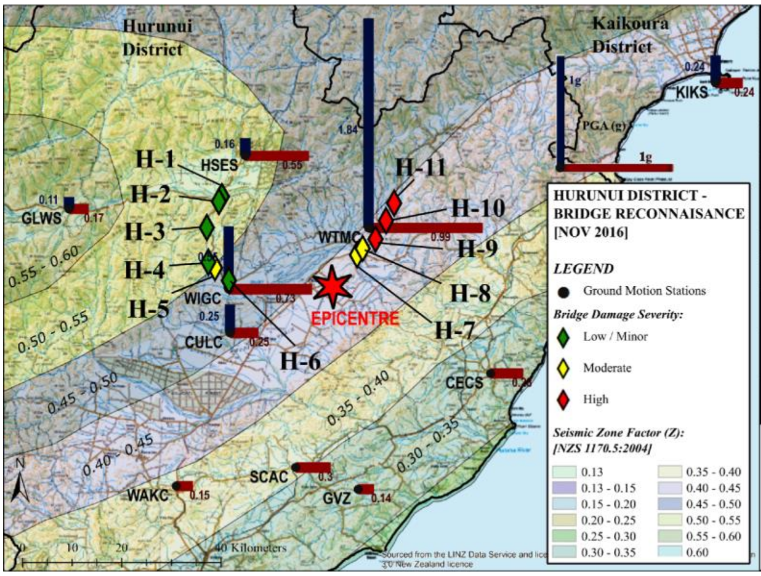
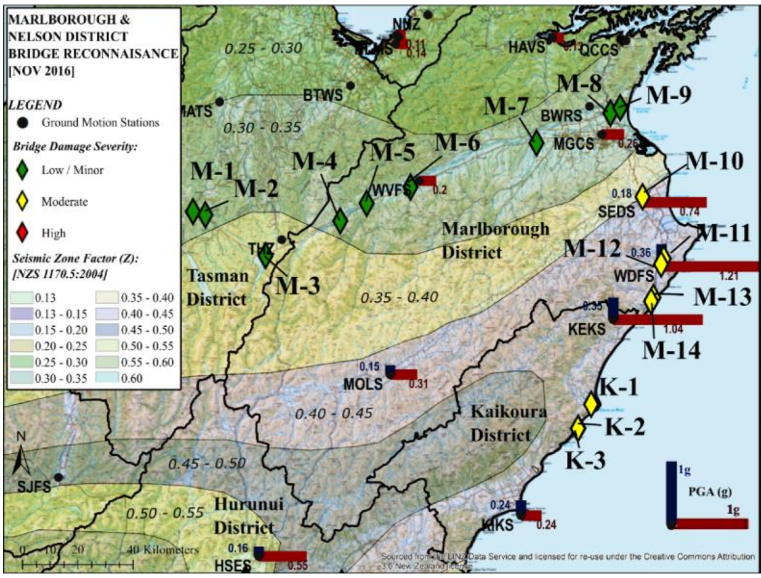


ID	Bridge Name	Route
H1	Hanmer River Bridge	SH7A
H2	Waiau Ferry Bridge	SH7A
H3	Glenallan Stream Bridge	SH7
H4	Brown Stream Bridge	SH7
H5	Stinking Stream Bridge	SH7
H6	Leslie Road Bridge	Leslie Hills Road
H7	Waiau River Bridge	SH70
H8	River Road Bridge	River Road
H9	Lower Mason Bridge	Inland Road
H10	Lottery River Bridge	Inland Road
H11	Wandle River Bridge	Inland Road

Figure 1: Map of bridges inspected in the Hurunui District with maximum measured PGA (vertical and horizontal data sourced from [5]) and NZS 1170.5: 2004 Hazard Factor contours overlaid.



ID	Bridge Name	Route
M1	Hope River Bridge	SH63
M2	Buller River Bridge	SH63
M3	Black Valley Stream Bridge	SH63
M4	Wairau River Bridge	SH63
M5	Branch River Bridge	SH63
M6	Wye River Bridge	SH63
M7	Waihopai River Bridge	SH63
M8	Ferry Road Bridge	Ferry Road
M9	Neal Road Bridge	Neal Road
M10	Awatere River Bridge	SH1
M11	Flaxbourne River Bridge	SH1
M12	Needles Creek Bridge	SH1
M13	Waima River Bridge	SH1
M14	Waima Overbridge	SH1
K1	Clarence River Bridge	SH1
K2	Mororimu Overbridge	SH1
K3	Mororimu Stream Bridge	SH1

Figure 2: Map of bridges inspected in the Kaikōura and Marlborough Districts with maximum measured PGA (vertical and horizontal data sourced from [2]) and NZS 1170.5: 2004 Hazard Factor contours overlaid.

SEISMIC DEMAND

There was large spatial variation in shaking intensity not just between each district but also within each district Figure 1 and 2 show the distribution of maximum horizontal and vertical peak ground accelerations; location and damage levels of the bridges inspected during the reconnaissance; and the Seismic Hazard Factors according to NZS1170.5: 2004. The figures confirm that as expected more severe damage occurred where shaking intensity was greatest.

The highest levels of structural damage occurred near Waiau Township in the Hurunui district close to the epicentre of the earthquake. Seven moderately damaged bridges were located on SH 1S stretching from Okiwi Bay to just north of Seddon (SEDS instrument station).

Judging the level of damage based on shaking intensity is inadequate because it does not indicate the spectral characteristics of the ground motion experienced by the bridges or by how much each bridge's design capacity was exceeded. Both of which are important points to be known given the large range of ages of affected bridges, the variation in dynamic properties between bridges, and most importantly, the variation in design philosophy and seismic detailing which can dictate either desirable or undesirable performance of the bridges during an earthquake. Figures 3, 4 and 5 show the geometric mean (of the two horizontal components) of the 5% damped elastic horizontal pseudo-spectral accelerations measured in Waiau (WTMC station: Te Mara Farm Waiau), Ward (WDFS station: Ward Fire Station) and Kaikōura (KIKS station: Kaikōura Strong Motion Station).

Overlaid on Figures 3, 4 and 5 are the NZ elastic design spectra used during three design eras: 1930's to mid 1960's, 1965 to 1987, and 2004 to present. The present day elastic design spectra was taken from NZS1170.5:2004. It was obtained for a Bridge Manual Importance Level 2 (1/1000-year annual probability of exceedance, ULS) structure, sited on soil corresponding to the station locations (soil class C: WTMC [6]; soil class D: WDFS [6], and soil class B: KIKS [6]), and having a Hazard factor Z = 0.45, with no near fault effects. The two oldest design spectra (based on working stress design) were taken from [1] and modified for compatibility with limit state design. In terms of the spectral characteristics of the measured shaking, it can be seen that at the WTMC and WDFS stations very large short period accelerations far exceeding the NZS1170.5 1/1000 year annual probability of exceedance design spectra occurred. At WTMC the spike in the geometric mean pseudo-acceleration (3.14g) occurred at a period of 0.29s. In contrast, the shaking observed in Kaikōura was well below the 1/1000 annual probability of exceedance design spectra with the shape of the spectral curve being relatively constant for periods larger than about 0.75s. The Kaikōura recording station is located on rock and this would be one of the reasons for the reduced spectral accelerations. The spikes seen in the short period range for the WTMC and WDFS spectra are most likely related to near fault effects as some of the recording stations were sited very close to locations of observed surface fault rupturing (Figure 6).

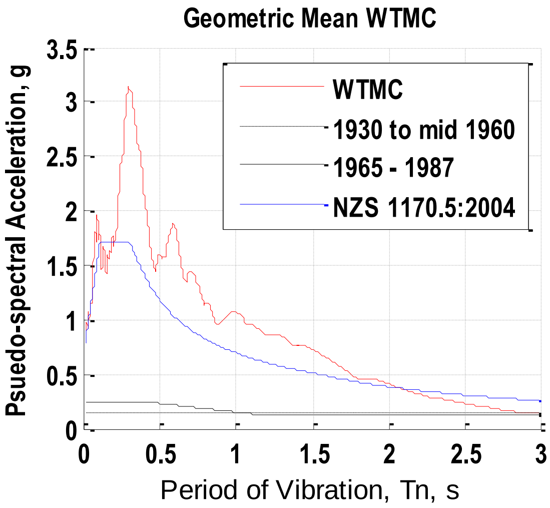


Figure 3: Geometric mean of the pseudo-spectral acceleration measured at WTMC strong motion station.

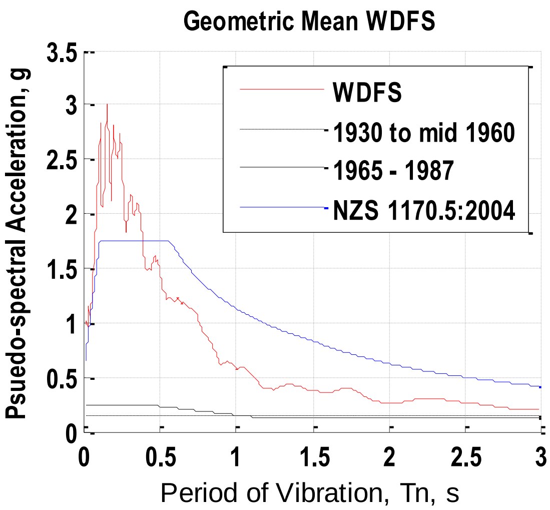


Figure 4: Geometric mean of the pseudo-spectral acceleration measured at WDFS strong motion station.

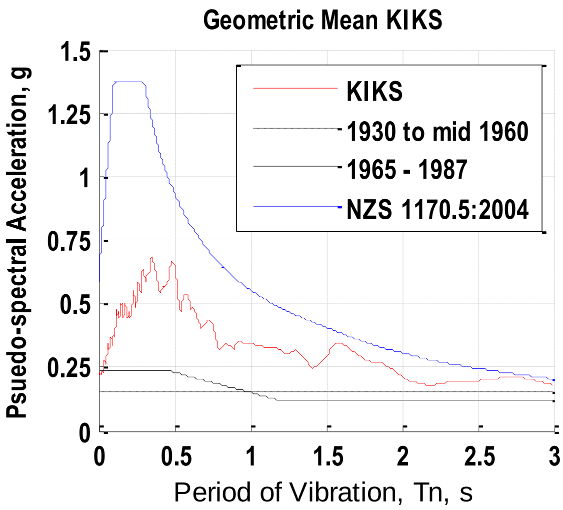


Figure 5: Geometric mean of the pseudo-spectral acceleration measured at KIKS strong motion station.

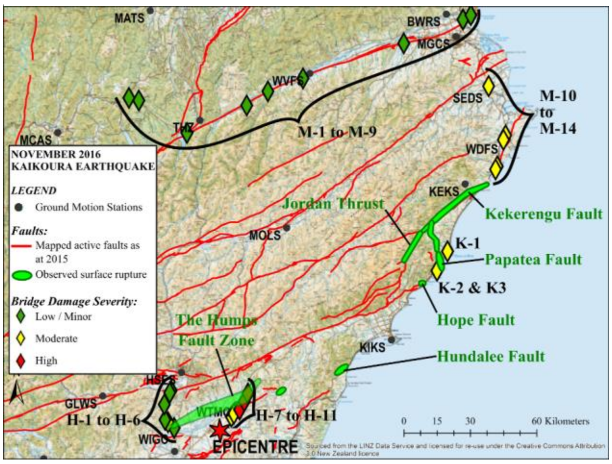


Figure 6: Map of inspected bridges overlaid with surface fault rupture observations based on reference [7].

Across all elastic spectral plots the two oldest design spectra were greatly exceeded. Many of the bridges inspected after the earthquake would have elastic periods of vibration within the range of 0.2 s to 1.0s which is coincidently the period of vibration where ground shaking at Waiau and Ward greatly exceeded the modern design spectra (Figure 3 and 4). In the Hurunui district the bridges inspected had a transverse period of vibration in the range of 0.2s to 0.6s and the pseudo-spectral acceleration experienced by these bridges greatly exceeded 1.5g. Whilst in the Marlborough region, for the bridges in the vicinity of the WDFS strong motion station, had a period of vibration in the transverse direction less than 0.2s due to those bridges using stiff wall piers, leading them to be subjected to a horizontal pseudo-spectral acceleration greater than 2g.

Another interesting point regarding the seismic demand imposed by the Kaikōura Earthquake is the extremely high vertical accelerations in Waiau and Ward (WTMC and WDFS in Figure 1 and 2). The vertical peak ground acceleration (PGA) in Waiau was measured to be 2.7g [8] and damage linked to high vertical accelerations was observed and will be elaborated upon further in this paper.

HURUNUI DISTRICT

A total of 11 bridges were inspected in the Hurunui district: one at the intersection of Leslie Hills road with SH7; three on SH7 just north of Culverden; two on SH7A near Hanmer Springs holiday park; one on SH70 at the southern entrance of Waiau Township; one on River Road at the north-western entrance (Leslie Hills road direction) of Waiau Township; and three on the Inland Road (linking Waiau to Kiakoura) between Waiau and Mount Lyford. Apart from the Stinking Stream Bridge on SH7, most of the bridges west of the epicentre (Figure 1) along SH7 and SH7A did not show any significant signs of earthquake damage. However, the bridges northeast of the epicentre (Figure 1) close to Waiau Township and along the Inland Road suffered moderate to high levels of earthquake damage.

The three bridges inspected on SH7 are single span bridges, of which the Stinking Stream Bridge and the Brown Stream Bridge have precast deck beams, while the Glenallan Stream Bridge is a cast-in-situ arch bridge. The Stinking Stream Bridge sustained moderate damage due to lateral spreading effects, causing approach settlement, and displacement of the wing walls along with minor cracking; which led to a speed restriction being placed on the bridge. The other two bridges had minor cracking, but were mostly unaffected in terms of operations.

The other bridges west of the epicentre were not seriously affected either. The Leslie Hills Road Bridge was unaffected by the earthquake and is a two span steel composite deck bridge. Of particular note with this bridge is that the span over the river channel has an obvious sag which could lead one to believe that the deck buckled due to lateral spreading. However, it is the authors understanding that the distortion of the deck existed prior to the earthquake due to a pier being lost in a flood. Similarly, the Waiau Ferry Bridge on SH7A, a single span steel truss bridge, was unaffected by the earthquake. However, the Hanmer River Bridge located further north on SH7A, a multi-span bridge with precast concrete beams supported on wall piers, did suffer damage at the wingwalls due to lateral spreading along with minor approach settlement. Damage caused by pounding at the abutments was observed along with some displacement at the seals, joints and bearings, due to transverse ground shaking. Nevertheless this bridge was open without any restrictions. Figure 7 shows a selection of photos of the bridges west of the epicentre.

The bridges inspected north east of the epicentre include: Waiau River Bridge, Mason River Bridge, Lower Mason River Bridge, Lottery River Bridge, and the Wandle River Bridge. The damage sustained to these bridges was substantial and will be described in detail below according to design era.

Early Seismic Standards – 1930's to Mid 1970's

The single-lane Waiau River Bridge (Figure 8) was built in 1965. The bridge is a thirty three span, simply supported, precast beam bridge with five, 17m, I - Beam units making up the superstructure. The superstructure is supported by seat abutments and thirty two wall piers typically 4m wide but 7.25m wide at each of the two passing bays. The beams and diaphragms between them sit on 12mm thick full-width neoprene strip bearings at each of the supports, and is restrained by dowels holding the end diaphragms of the deck at the piers and abutments.

Damage sustained by the Waiau River Bridge was moderate, with the bridge being operated under speed restrictions. The second and third spans of the bridge on the west side was observed to have been rotated in plan view (Figure 8c). This was clearly visible from the closed expansion gaps at pier 3 (upstream side), which showed slight signs of transverse rotation towards the upstream side. However, there was no sign of localized pounding damage despite the residual deck displacements. Also, the abutments showed signs of rotation along with some soil subsidence; pile exposure was observed at the eastern end and extreme abutment cracking at the west end of the bridge (Figure 8d) Most piers showed cracks at the pier-pile cap interface with some minor spalling, while at some piers near the waterway, longitudinal bar exposure and buckling was observed as well (Figure 8e)

(a) Glenallan Stream Bridge SH7

(b) Stinking Stream Bridge SH7

(c) Waiau Ferry Bridge SH7A

(d) Hanmer River Bridge SH7A

Figure 7: Bridges inspected west of the epicentre.

(a) Oblique View

(b) View from top of deck

(c) Rotation of span 2 and 3

(d) Cracks and rotation on the western abutment

(e) Cracking and buckling at the base of pier

Figure 8: Waiau River Bridge SH70 and damage observed at the Waiau River Bridge.

From the preliminary performance observations, the Waiau River Bridge structure seems to be quite a rigid structure in the transverse direction, due to the wall pier stiffness. Strong ground shaking was experienced; however this did not damage the structure significantly apart from the deck gap openings at the passing bay transitions and bar buckling at the base of the piers due to the lack of transverse tie reinforcement. The liquefaction ejecta observed at the site in the mid and the eastern end of the bridge was relatively minor, but apparently did not affect the structure. The deck rotation observed was mainly due to the transverse rotation of pier 3 possibly caused by combined effect of settlement and ground shaking. The damage at the approaches and abutments was more significant due to soil subsidence and lateral spreading effects that caused large cracks in the soil embankments. The observations indicated a good overall structural performance of the bridge.

Early Ductile Standards – Mid 1970's to Late 1980's

The Mason River, Lower Mason River, Lottery River, and Wandle River bridges (Figure 9) were constructed in the period ranging from 1980 to 1987. All of these bridges are structurally similar to each other. They all are multi-span precast concrete bridges supported on single column piers with hammer head pier caps, and are composed of twelve, eight, six and three spans, respectively. The Lottery and Lower Mason River bridges have simply supported 20m precast I-beams, while, the Mason River and Wandle River bridges have 16-18m precast double hollow core beam units.

The Lower Mason (Figure 9b) and Lottery River (Figure 9c) bridges are identical in the way they resist lateral loading. These structures have been seismically split in two: longitudinal interaction between each half of the bridge has been isolated by a movement joint with knock off detail, whilst, transverse interaction has been isolated by the omission of transverse shear keys at the central pier. In addition to this, these bridges use a rudimentary form of seismic isolation under transverse loading, whereby there is a large gap between the deck beams and the transverse shear keys such that they may displace an appreciable amount on the supporting elastomeric bearings before contacting the shear keys. This design approach therefore makes use of the low lateral stiffness of the bearings to elongate the period of the structure. There are no transverse shear keys at the abutments. In the longitudinal direction, lateral loads are directly transferred to each of the piers through bearing of the deck diaphragms on concrete upstands working as shear keys. Also, the deck beams are tied together longitudinally by tight linkages and a continuous deck between spans, except, over the central pier There are no longitudinal linkages at the abutments but the beams rest on wide seats. Therefore, all piers resist longitudinal lateral loading, except the central pier which is effectively much more flexible having no shear keys. Transverse load is transferred to all piers through the bearings but under large displacement response, when the beams contact shear keys, the central pier has lower stiffness than the other piers and will carry lower loads than the others. Until the beams contact the shear keys the overall transverse response can be thought of as rigid body translation of each half of the superstructure.

The Mason River Bridge (Figure 9a) is split in two with a central movement joint similar to the Lower Mason and Lottery River bridges. There are transverse shear keys at the abutments and "tight" internal longitudinal shear keys and linkage bars over all of the piers except the central pier At the central pier, provision is made for longitudinal movement with loose linkage bars but steel plate lateral support stubs cast into the piers restrain the transverse movement. Therefore, the overall transverse response can be thought of as the deck having a curved displaced shape where the largest transverse displacement occurs at the central pier with the abutments providing a "pin" restraint.

The Wandle River Bridge (Figure 9d) is curved in plan and has a slope in the longitudinal direction. Linkages, shear keys and elastomeric bearings restrain the transverse and longitudinal movement over the piers, but only linkage bars resist both the transverse and longitudinal movement at the abutments. Longitudinal movement towards the backfill at the abutments is resisted by the abutment back-wall which is cast against the beams with a 6mm separation gap.

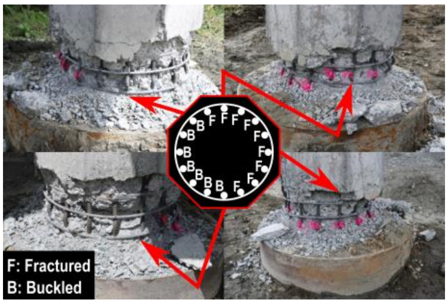
(a) Mason River Bridge – River Road

(b) Lower Mason River Bridge - Inland Road

(c) Lottery River Bridge - Inland Road

(d) Wandle River Bridge – Inland Road

Figure 9: Bridges designed during the era of early ductile standards in the Hurunui district.



(a) Plastic hinge zone and fractured bars – Lower Mason

(b) Plastic hinge zone - Lottery River

(c) Plastic hinges in piers - Mason River

(d) Plastic hinge and spalling - Wandle River

(e) Tilting of pier - Wandle River

Figure 10: Pier damage sustained by Hurunui district bridges designed to the early NZ ductile standards.

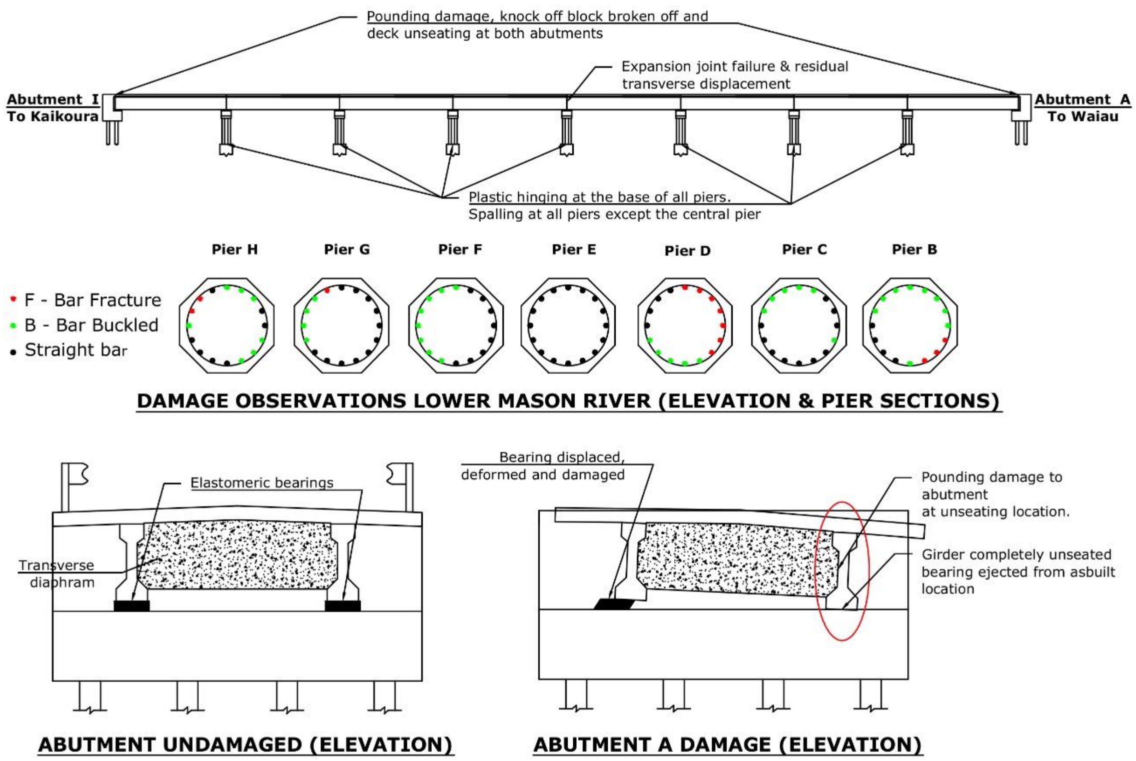


Figure 11: Damage schematics for the Lower Mason River Bridge.

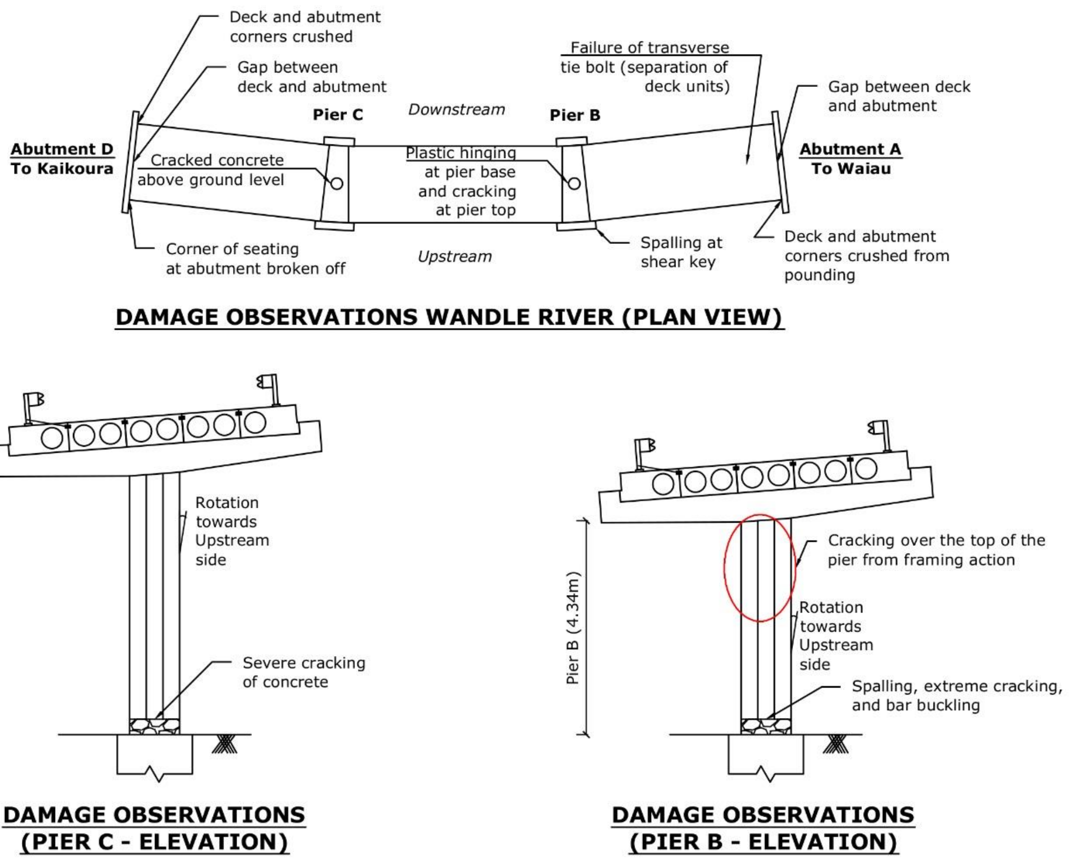


Figure 12: Damage schematics for Wandle River Bridge.

The Lower Mason and Lottery River bridges suffered similar damage. Observed damage common to both bridges included: damage to the knock-off block at the central pier movement joint (Figure 13a); failure of the abutment knock- off blocks; residual displacement of most of the elastomeric bearing pads at the piers and abutments; and residual displacements at the central pier and abutment movement joints. Unseating of the deck beams only occurred at the Lower Mason River Bridge (Figure 13b), where, at each abutment one deck beam (on opposite sides) was observed to be unseated off the supporting elastomeric bearing. The piers suffered extensive damage at both bridges, where all piers except the central one formed fully developed plastic hinges at the base. Extensive cover spalling, and a significant number of buckled and fractured longitudinal bars were seen at the Lower Mason River Bridge (Figure 10a and 11). Pier damage at the Lottery River Bridge was similar but there was no evidence of bar fracture (Figure 10b).

Similar damage observations were made for the Mason River Bridge, where the deck joints had opened or closed, and pier plastic hinges with bar buckling and concrete spalling were observed with increasing degree of damage towards the centre pier (Figure 10c). The pier damage pattern was interesting at this bridge as concrete spalling and rebar exposure occurred mainly on the south side of the piers (transverse loading direction) and tended to be more in the longitudinal loading direction close to the abutments.

Extensive damage was observed to the Wandle River Bridge (Figure 12) which included: separation between the hollow core deck units at the south end of the bridge; translation of deck units in the downstream direction; hinging and rotating of piers towards the upstream side (in the direction of increasing radius of curvature); opening of the deck joints at the abutments, along with significant approach damage (Figure 10d and 10e).

The preliminary performance analysis on the Lower Mason River and Lottery River bridges indicates that both bridges were highly loaded both in the transverse and longitudinal directions with the damage level exceeding the life safety performance limit causing the piers to develop plastic hinges in all piers with less severe hinging observed at the central pier of the Lower Mason bridge. All of the knock-off devices at the movement joints were damaged. It is probable that the knock-off blocks at the abutment movement joints failed first, increasing the displacement demand of the bridges and creating pounding of the abutments. This was probably followed by failure of the central movement joint. Finally, this situation led to an asymmetric unseating of the beams from their bearings at the abutments of the Lower Mason Bridge. Bearing movement was observed at the Lottery River Bridge, but, no unseating occurred. Also, the abutments were subjected to lateral spreading of the soil and probably some slight settlement.

The same conjectured response can be extended to the Mason River and Wandle River bridges, as most of the damage was purely due to ground shaking. The piers performed well, as they developed plastic hinges at the pier bases and went through the expected failure mechanism. However, significant instability was visible in the curved Wandle Bridge, where most likely all orthogonal motions were interacting with the bridge structure, causing large displacements (leading to probable failure of transverse tie-rods, if present) and separation gaps between some of the deck beam units. It is also believed that the excessive residual tilt of the piers is not only due to plastic hinging but may also result from the asymmetric transverse stiffness (from the curved plan layout), causing ratcheting in the structure. The layout results in greater transverse stiffness in the downstream direction due to arching action between the abutments, relative to the upstream direction, where the bridge is only restrained by linkage bars at the abutments; leading to hinging and residual tilts towards the less stiffer upstream direction. Additional contribution to the tilt may also be possibly due to failure of the foundations due to the pier piles being quite short. However, the piles are founded in rock so residual displacement from the piles was probably small.

(a) Compression failure of the expansion joint over the central pier

(b) Residual displacement of bearing (left in image), unseating from elastomeric bearing (right in image)

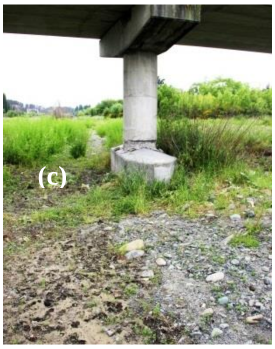
Figure 13: Observed superstructure damage at the Lower Mason River Bridge.

Geotechnical Observations

Most of the bridges in the Hurunui District suffered from approach settlement, resulting in minor to moderate disturbances in operation of the bridges. Almost all bridges northeast of the epicentre had gravel fill added after the earthquake to the approaches to accommodate these significant approach settlements. Evidence of liquefaction and lateral spreading were observed in the bridge surrounds at most of the surveyed sites. The typical damage observations related to these geotechnical phenomena included minor rotation in the abutments, subsidence in the approach embankment zones, and relatively minor approach and pavement damage in the form of longitudinal and transverse cracking. Even though there were significant signs of liquefaction and lateral spreading around the bridges inspected, this did not seem to cause any direct, significant, impact on the structural integrity or operation of the bridges themselves beyond the approach settlements which were easily addressed. Figure 14 shows some typical examples of the geotechnical damage observed at the bridges inspected in the Hurunui district

(a) Embankment failure due to lateral spreading (Hanmer River Bridge)

(b) Embankment failure due to lateral spreading (Waiau River Bridge)



(c) Liquefaction ejecta near central pier. (Mason River Bridge)

(d) Ground offset with foundation (Mason River Bridge)

Figure 14: Geotechnical observations in the Hurunui District, within bridge vicinity.

Non-Structural Elements and Utilities

Guardrail damage was seen at the Waiau, Lower Mason, and Lottery River bridges (Figure 15a-d). At the Waiau River Bridge this was due to deck rotation; at the Lower Mason River Bridge, it was due to approach settlement and compression failure of the movement joint over the central pier; and at Lottery River Bridge, it was due to lateral deck offset and approach settlement (Figure 16a-c). Services were located on Stinking Stream, Waiau Ferry, Hanmer River, Waiau River, Lower Mason River and the Lottery River Bridges. However, damage was only observed at three of these bridges. At the Hanmer River Bridge, the telecom cable was observed to have been stretched and ruptured along with damage to the duct at both ends. The water pipes at the Lower Mason River Bridge had ruptured at the abutments and the central expansion joint, and at the time of the reconnaissance had already been repaired at those sections. The telecom cable duct on the other side of the bridge was observed to have broken at the abutments. At the time of survey, the pipes at the Lottery River Bridge were intact but there was evidence that minor pipe repairs had been carried out at different sections, most likely related to earthquake damage given the amount of residual deck displacement observed.

(a) Waiau River Bridge

(b) Lower Mason River Bridge

(c) Lottery River Bridge – approach

(d) Lottery River Bridge – deck offset

Figure 15: Observed guard rail failures.

(a) Telecom cable (Hanmer River Bridge)

(b) Water pipe (Lower Mason Bridge)

(c) Telecom line (Lower Mason Bridge)

Figure 16: Services damage at Hurunui district bridges.

KAIKŌURA DISTRICT

In the Kaikōura District, a total of three road bridges were inspected: Mororimu Stream, Mororimu Overbridge, and Clarence River Bridge. All three are on SH1 north of Kaikōura. At the time of the reconnaissance, the southernmost bridge which could be accessed by SH1 north of Kaikōura was the Mororimu stream bridge. This was due to substantial landslides at the southern end of Okiwi Bay (25km north of Kaikōura) blocking road access further south (Figure 17).

(a) Okiwi Bay *(b) Close up of slip*

Figure 17: Landslide at the southern end of Okiwi Bay.

Early Seismic Standards – 1930's to Mid-1970's

The Mororimu Stream Bridge and Mororimu Overbridge (Figure 18) were constructed in 1951. These bridges are cast in-situ integral structures and are supported on piers with strip footing foundations. They are close in proximity to one another, being located only about 100m apart. Other than these similarities there are many differences between the two structures: the Mororimu Stream Bridge has 3 spans, whilst the overbridge has 4 spans; the stream bridge uses wall piers, while, the overbridge uses multi-column bents (4 column bents for the outer spans and a portal frame for the central pier); and the stream bridge is both longitudinally sloped and curved horizontally, while, the overbridge is level and straight.

(a) Mororimu Stream Bridge

(b) Mororimu Overbridge

Figure 18: General view of the Mororimu bridges.

The structural damage observed at the Mororimu Stream Bridge was mainly confined to the substructure, with only one transverse crack found at the underside of the deck on the stream side of the western pier (Figure 19). Both of the abutments were extensively cracked along with the tops of the abutment columns. Horizontal cracking at the tops of the piers was also observed (Figure 19c).

The distribution of damage at the Mororimu Overbridge was different than at the stream bridge, because the pier system was column bents instead of wall piers. Cracking was observed at the tops of the abutment columns as well as the tops of the columns at each of the pier bents (Figure 20). At the central pier (a two column portal frame bent), cracking was extensive around the knee joints of the portal frame bent (Figure 20c). Concrete spalling tended to occur at the tops of the shortest columns at each of the bents resulting in exposed reinforcing (Figure 20d).

(a) Abutment cracking *(b) Abutment column cracking*

(c) Horizontal cracking at the top of a pier

(d) Transverse cracking of the soffit near a pier

Figure 19: Damage at the Mororimu Stream Bridge.

(a) Abutment column cracking *(b) Spalling at the top of the column bent*

(c) Cracking below knee joint of central pier *(d) Close up of exposed bar at the top of a column*

Figure 20: Damage at the Mororimu Overbridge.

Early Ductile Standards – Mid 1970’s to Late 1980’s

The Clarence River Bridge (Figure 21), built in 1975, seismically strengthened in 2007, and was the most significant structure visited in the Kaikōura District. It is a 6 span, balanced cantilever single cell box girder structure. Each pier and the half span on either side acts as a cantilever connected to its neighbour by an expansion joint and linkages. The superstructure is supported by wall piers and is monolithic with the piers. In terms of foundations, the piers are each supported by two 2.4m diameter concrete shell piles under-reamed at the pile base, whilst the abutments are supported by steel H piles. At the abutments, the box girder is seated on three square elastomeric bearings. A row of 6 hold-down rods anchored on the inside of the bottom flange of the box girder prevent uplift of the end spans under live loading. This bridge has a complex geometry, being longitudinally sloped and moderately curved in the horizontal plane (concave side facing east).

Figure 21: General view of Clarence River Bridge.

(a) Abutment cracking

(b) Hold down rod residual deformation

(c) Deck settlement

(d) Concrete spalling at pier base

(e) Damaged pile cap

Figure 22: Observed damage at the Clarence River Bridge.

Most of the damage observed at the Clarence River Bridge occurred at the substructure level. The northern abutment suffered cracking of the west wing wall and vertical cracks at the joint between the back wall and transverse shear key on the east side of the abutment (Figure 22a). The southern abutment was undamaged. Of the 5 piers, only the 2nd pier from the northern end of the bridge was observed to be damaged. Concrete spalling around the pier base was observed (Figure 22d) in addition to a large diagonal crack in pile cap (Figure 22e). Similar cracking damage was reported in the pile caps at a number of the other piers. The diagonal crack in the pile cap implies a vertical punching shear failure had begun to manifest due to the earthquake shaking. At the southern abutment, the vertical hold downs at the tip of the cantilever span had pulled out of the bottom of the box girder causing concrete spalling (Figure 22c). Based on the appearance of a reduced clearance between the abutment seat and the deck soffit the deck appeared to have dropped vertically. At the northern abutment, the vertical hold down rods did not pull through the box girder but were pressed against one side of the surrounding duct (Figure 22b) indicating that the deck had a residual displacement towards the east.

(a) Approach settlement (Mororimu Stream)

(b) Abutment fill settlement (Clarence River)

(c) Soil gapping (Clarence River)

(d) Soil gapping (Mororimu Overbridge)

(e) Lateral approach spread (Clarence River)

Figure 23: Geotechnical damage observed at surveyed Kaikōura district bridges.

Geotechnical Observations

Approach settlement was observed at all bridges examined in this district (Figure 23a). At the Mororimu Stream and Overbridge, the settlement was extreme, being measured to be 100mm. At the Clarence River Bridge, the presence of settlement slabs lessened the direct impact of approach settlement so that there was no abrupt step between the approach level and deck level, however, settlement at the northern end of the bridge was large enough to cause a 25mm wide crack in the asphalt where the approach slab meets the abutment. Settlement and slumping of fill below the abutment pile caps and footings (Figure 23b) was observed at all bridges in addition to soil gapping around some of the piers (Figure 23c and d). A severe case of transverse approach spreading was also observed at the southern end of the Clarence River Bridge (Figure 23e). The geotechnical damage does not appear to be due to liquefaction due to the lack of ejecta and lack of fracturing of the ground surface.

Non-Structural Elements and Utilities

Road surface damage in the form of fissures, cracks, or breaks was observed. None of the bridges described in this section carry utilities.

MARLBOROUGH DISTRICT

In the Marlborough district, a total of 11 bridges were inspected: four on SH63 south of Blenheim, two single lane local road bridges approximately 7km northeast of Blenheim; and five SH1 bridges south of Blenheim. The bridges inspected on SH63, along with the two local road bridges north of Blenheim did not show any sign of earthquake damage. The majority of these bridges were multi-span, simply supported reinforced concrete bridges with mono pier-pile hammerhead piers. Figure 24 contains a selection of photos showing the undamaged bridges The SH1 bridges in the Marlborough district inspected at the time of the reconnaissance were the Awatere River Bridge, Flaxbourne River Bridge, Needles Creek Bridge, Waima River Bridge and Waima Overbridge. All of these bridge were found to have sustained earthquake induced damage.

(a) Wairau River (Wash) SH63

(b) Wye River SH63

(c) Ferry Bridge, Ferry Road, Spring Creek

Figure 24: Selection of undamaged bridges west and north of Blenheim.

Early Seismic Standards – 1930's to Mid-1970's

The Flaxbourne River (Figure 25a) and Needles Creek (Figure 25b) Bridges were built in the 1950's and are both 5 span structures supported on wall piers. In the case of the Needles Creek Bridge, the structure is entirely integral unlike the Flaxbourne River Bridge where the superstructure is simply supported on the piers and is only connected to the piers by eight vertical, 32mm diameter dowel bars (two per beam).

The Flaxbourne River Bridge suffered damage to both the substructure and superstructure (Figure 26), whereas the Needles Creek Bridge mainly suffered substructure damage. A point of similarity between the Needles Creek and Flaxbourne River bridges in terms of structural damage was the separation of the deck from the piers and abutments due to excessive lateral loading. In the case of the Flaxbourne River Bridge, the vertical dowel bars connecting the deck to the piers and abutment were not sufficient to provide the horizontal shear strength necessary to withstand the lateral seismic demand from the deck. Over the piers, the vertical bars had broken the surrounding cover concrete, bent the confining transverse reinforcement and in some cases fractured (Figure 27a). Over the first pier from the southern end it appeared that all of these vertical bars had fractured. This damage was more severe on the first two piers at the southern end of the bridge. The deck had moved from its original position in a south west direction over these piers (Figure 27b). The two piers closest to the southern abutment appeared to be less damaged than the other two piers where the deck-pier connection was not as badly damaged. At the third pier from the south end of the bridge an angled crack was observed at the top of the pier indicating that the pier-beam connection had started to fail the concrete plinth in shear.

(a) Flaxbourne River SH1

(b) Needles Creek SH1

Figure 25: Flaxbourne River and Needles Creek Bridges.

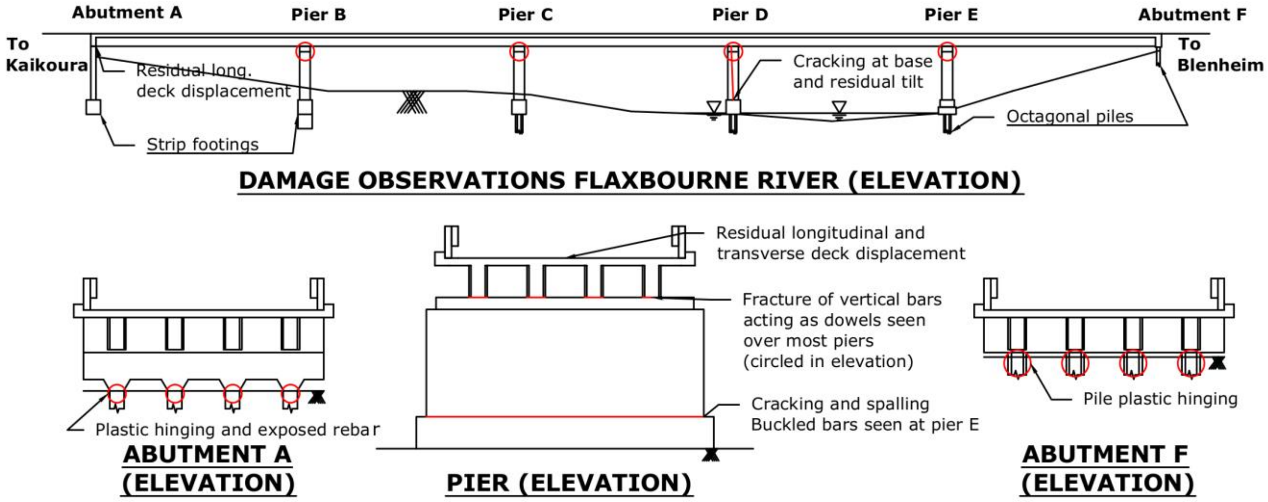


Figure 26: Damage schematics of the Flaxbourne River Bridge.

In terms of substructure damage at the Flaxbourne River Bridge, plastic hinging occurred in the foundations at both abutments as well as cracking with reinforcement exposure and buckling at the base of some of the piers. At the southern abutment, plastic hinging occurred at the top of the foundation columns that extend above the strip footing (Figure 27c); and at the other abutment, plastic hinges formed at the top of the piles. Concrete spalling was more significant at the south abutment, fully exposing the longitudinal reinforcement of the abutment columns.

(a) Damaged deck – pier connection

(b) Residual transverse displacement of the deck relative to the pier

(c) Exposed reinforcing of the abutment columns

(d) Residual longitudinal deck displacement

Figure 27: Abutment and superstructure damage at the Flaxbourne River Bridge.

The abutment columns were observed to use plain round bars with widely spaced stirrups. At the same abutment, the transverse capping beam on the columns had moved 200 mm towards the river. Also, there was clear signs of longitudinal movement of the beam diaphragm on top of the abutment seat (Figure 27d). At the north abutment, cracking of the piles extended across the depth of the pile with spalling occurring on both the front and back faces of the piles.

(a) Tilted pier

(b) Pier elevation showing base cracking

(c) Close up of crack at the pier base

(d) Buckled longitudinal reinforcement

Figure 28: Pier damage at the Flaxbourne River Bridge.

The pier-pile cap interface was only able to be accessed at the three piers closest to the northern end of the bridge and cracking at the pier base was observed at all of these piers. The same piers were observed to be tilted in the longitudinal direction such that the tops of the piers had moved south (Figure 28a). At the base of the piers a single crack was observed to extend through the entire pier-pile cap interface (Figure 28b). Exposure and buckling of longitudinal bars was observed for the two northernmost piers (Figure 28c and d). At the northernmost pier there was evidence that the pier had slid on the crack interface approximately 10mm towards the river channel.

At Needles Creek Bridge, significant plastic hinging at the top and bottom of the piers occurred. However, single cracks instead of distributed cracks tended to form (Figure 29a). The cracks were particularly wide at the two piers at the north end of the bridge. At the pier closest to the north abutment, exposure and buckling of the longitudinal bars was evident at the top of the pier. This pier also had a residual tilt of about 3° from vertical (Figure 29b). The manner of the tilt was such that the bottom of the pier D appeared to have moved towards the waterway. The mode of cracking of the piers and abutments (over the entire width of the bridge) (Figure 29d) indicates that lateral spreading effects were significant at this bridge [4].

(a) Typical cracking at the top of the piers

(b) Tilted pier

(c) Cracking at the top of the abutment piles

(d) Cracking of the abutment-deck connection

Figure 29: Observed structural damage at the Needles Creek Bridge.

Early Ductile Standards – Mid 1970's to Late 1980's

The Waima River (Ure) (Figure 30a) and Waima Overbridge (Figure 30b) were constructed in 1975 and 1985, respectively, and are located just over 1km apart on SH1. The Waima river bridge has eight simple spans and uses two precast prestressed I beams seated on 1.5m diameter, circular, mono-pile, hammerhead piers. This bridge was seismically retrofitted in 2003 with external steel shear keys to prevent unseating of the deck (Figure 31a). The Waima Overbridge, is a corrugated steel multi-plate arch tunnel approximately 114m long and having a diameter of 6.75m.

The Waima River Bridge was one of the few bridges visited in the Marlborough district which displayed noticeable residual transverse displacement of the deck. The residual deck displacement had two components, a rigid body translation towards the east and rotation in the transverse west direction between the abutments with the maximum transverse displacement of the deck being near the middle of the bridge. The deck was also twisted about its longitudinal axis with the southbound lane higher than the northbound lane. Cracking and spalling of concrete was observed around the retrofitted steel brackets restraining longitudinal movement of the precast I-beams (Figure 31c). In terms of damage to the substructure, cracking was observed at both abutments in addition to cracking of the top of the abutment piles (Figure 31b). The second (Figure 31d) and third piers from the southern abutment were observed to have noticeable tilting westward. Flexural cracking was observed around the base of the second pier from the southern end of the bridge just above water level at the edge of the waterway, and measured to be approximately 1mm wide. At the time of reconnaissance, only pier damage above ground and water level could be observed and appeared to be minor. However, investigation by Opus International Consultants after the reconnaissance (Figure 33) show that pier plastic hinging had occurred below ground level and that only cracking of the concrete occurred within the plastic hinge region.

(a) Waima River (Ure) Bridge SH1

(b) Waima Overbridge SH1

Figure 30: General views of the Waima River and Overbridge Bridges.

(a) View of the south abutment

(b) Cracking of the pile – abutment interface

(c) Spalling of concrete around the retrofitted steel brackets attached to the beams

(d) Slight tilting of the first pier from the southern abutment westward

Figure 31: Structural damage observed at the Waima River Bridge.

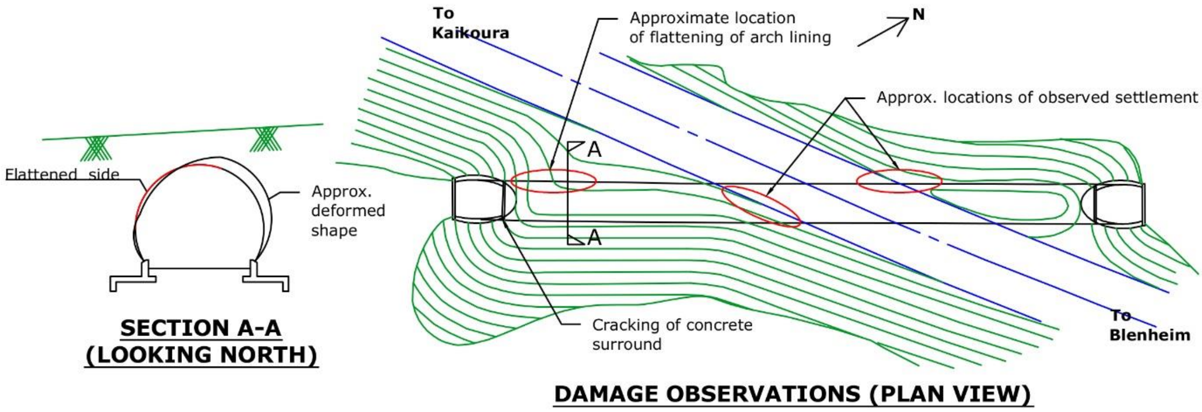


Figure 32: Damage schematics for the Waima Overbridge.

The damage observed at the Waima Overbridge was mainly geotechnical in nature (Figure 32). This is described in the following geotechnical damage section. The main structural damage observed was a flattening of the west side of the arch at the southern end of the structure (Figure 34a) and cracking and spalling of the concrete surround at the southern end of the overbridge (Figure 34d) in proximity of the concrete foundation. The flattening of the arch was large enough to crack the paint protective coating on the corrugated steel section (Figure 34b and 34c).

Figure 33: Typical pier damage observed after excavation and dewatering [courtesy of OPUS International Ltd].

(a) Flattened southern end of overbridge

(b) Close up of flattened side of arch lining

(c) Cracked surface coating at flattened part of the arch lining

(d) Cracking of concrete surround at the southern end

Figure 34: Observed damage at the Waima Overbridge.

Current Standards – Post 2003

Of the bridges visited, the Awatere River Bridge (Figure 35) is the newest structure, being constructed in 2007. It is a 10-span structure having spans of approximately 27m. The superstructure consists of 1.2m deep precast pre-stressed U beams which are made integral with the piers buy casting in-situ diaphragms and is seated on elastomeric bearings at the abutment. The deck is continuous along the length of the structure and the substructure consists of pier bents made up of two 1m diameter, 5.5m long circular columns each supported on a single 1.2m steel cased drilled pile.

Figure 35: Awatere River Bridge looking north.

The Awatere River Bridge sustained flexural cracking at the top of the columns (the pier-pile interface was underground at most piers), at the piers near the abutments, and spalling of concrete at the top and bottom of the columns at the piers close to and at the middle of the bridge (Figures 36 and 37b-d). The bridge deck at the Kaikōura side abutment appeared to have a residual displacement in the southeast direction based on the residual displacement of the elastomeric bearings (Figure 37a). The cracking and spalling in the pier columns mainly occurred at locations where the concrete had been previously repaired with mortar after the 2013 Seddon earthquake (Figure 37d).

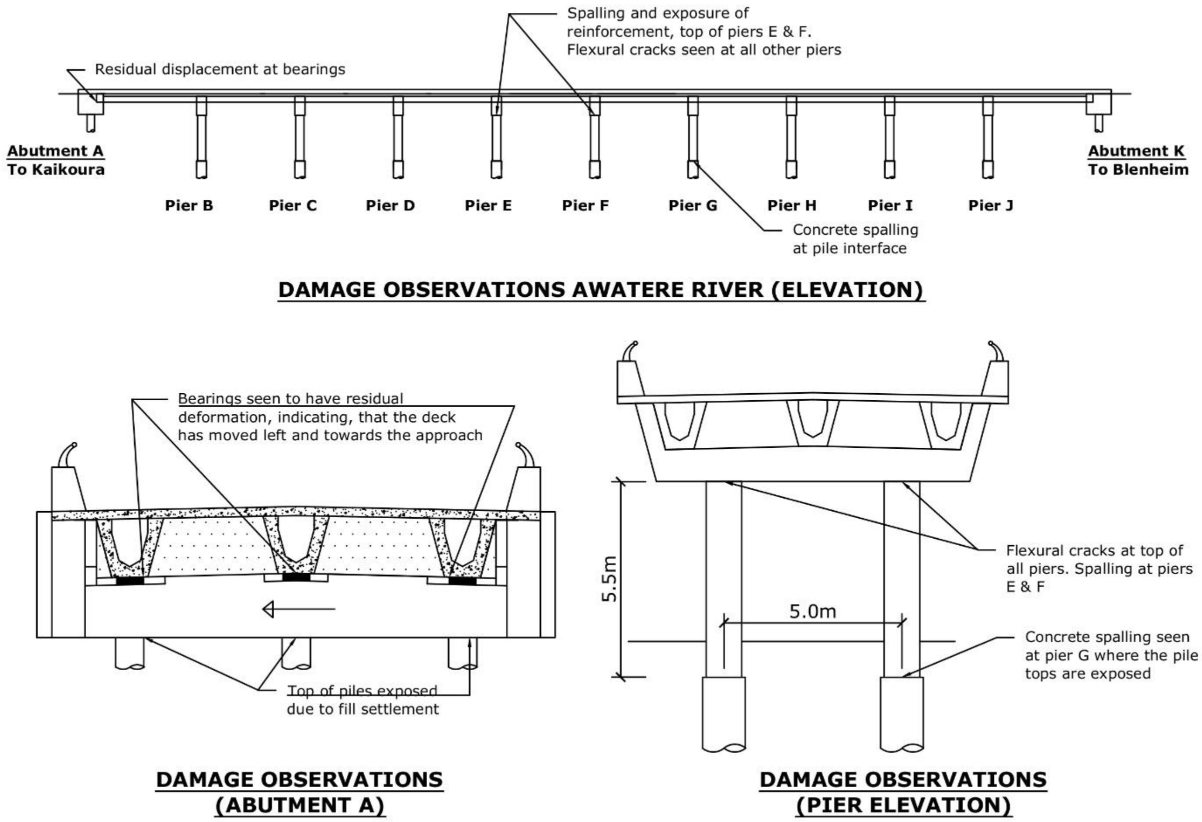


Figure 36: Damage schematics for the Awatere River Bridge.

(a) Bearing at Seddon end abutment showing residual deck displacement

(b) View of damaged pier bent near the middle of the bridge

(c) Concrete spalling observed at the base of the columns near the middle of the bridge

(d) Exposed reinforcing at the column top after spalling of the repaired concrete

Figure 37: Structural damage at the Awatere River Bridge.

Geotechnical Observations

Approach settlement was observed at all of the bridges except the Awatere River Bridge and Waima Overbridge. In the case of the Awatere River Bridge, it is to the author's knowledge that the abutments are of the Mechanically Stabilised Earth (MSE) wall type and that the lack of settlement is likely to be due to the soil reinforcement preventing spreading of the fill under earthquake shaking. The most severe case of approach settlement occurred at the Waima River Bridge (Figure 38b), 100mm settlement at the southern approach). Settlement of the fill on the abutment slopes beneath the bridges at the Awatere River (Figure 38d), Flaxbourne River, and Needles Creek bridges was also observed. In terms of superficial evidence of liquefaction, an isolated case of ejecta was observed around a few pier-piles of the railroad bridge parallel to the Waima River Bridge (approximately 50m upstream) as shown in Figure 38a. Lateral spreading cracks in the roadway were observed at the Waima River (Figure 38c), whilst, lateral spreading of the creek banks and abutment fill were observed at the Needles Creek bridge. A severe case of soil gapping measuring 100mm was observed at Needles Creek Bridge.

The damage which occurred at the Waima Overbridge was mainly geotechnical in nature. Significant soil settlement of the fill above the arch occurred on the seaward side at the southern end of the structure, resulting in a noticeable dip in the shoulder of the southbound lane (Figure 39). It is believed that the flattening of the tunnel lining on the southwest side was due to settlement of the fill on the southeast side causing unbalanced lateral earth pressure which the flexible lining could not sustain resulting in permanent deformation. More geotechnical details are reported in Stringer et al. [4].

(a) Liquefaction ejecta *(b) Large approach settlement (Orange spray paint)*

(c) Lateral spread crack at bridge approach *(d) Settlement of abutment fill*

Figure 38: Geotechnical damage observed in the Marlborough District.

Figure 39: Fill settlement and spread at the Waima Overbridge.

Non-structural elements and utilities

At the Flaxbourne River, Needles Creek, and Waima Overbridge the guardrails were damaged around areas where soil settlement and spreading occurred. The only observed damage to utilities was at Needles Creek Bridge where several small diameter plastic pipes may have been dislodged from attachments at deck level.

BRIDGE PERFORMANCE IN CONTRAST TO THE CANTERBURY EARTHQUAKE SEQUENCE (2010-11)

The Canterbury Earthquake Sequence (2010-11) (CES) resulted in strong ground shaking and ground deformation damage to various infrastructure in Christchurch, thereby setting up a benchmark in terms of damage observations, repair strategies, loss estimations, etc. The bridge performance contrast observed during the two earthquakes can be summarised as follows:

- Most of the damage observed to road bridges during the CES was attributed to liquefaction and lateral spreading effects [9]. Whereas, the dominant cause of damage during the Kaikōura Earthquake is mainly attributed to ground shaking intensity.
- The dominant damage mode during the CES was approach settlement and rotation of abutments due to liquefaction induced lateral spreading. Whereas, the dominant damage mode during the Kaikōura Earthquake was pier plastic hinging.
- In terms of structural characteristics, most of the bridges in Christchurch are short, wide, low in height, and of monolithic construction. Whereas, the bridges in the areas worst affected by the Kaikōura Earthquake are long, single / two lane bridges with relatively tall (and mostly column) piers. These differences in structural characteristics between bridges is another factor in the differences in observed damage between the bridges affected by the CES against those affected by the Kaikōura Earthquake. This is because pier flexural loading is much less for bridges with short piers in addition to the wide decks increasing the transverse stiffness of the bridge, reducing the transverse displacement demand on the piers.
- Damage to bridges in general was observed to be worse from the Kaikōura Earthquake, in contrast to the CES. However, most of the bridges were open (with restricted access and speed in some cases), apart from the Wandle River Bridge. This is in contrast, to the bridges in Christchurch; where some were closed for repair (a couple for more than a week), even though the bridges performed better [10].

CONCLUSIONS AND OPEN ISSUES

This paper provided a brief overview of the structural performance of bridges across the Marlborough, Hurunui and Kaikōura districts observed during the Kaikōura Earthquake. At the time of initial inspection, the bridges had been rapidly assessed and were generally only open to emergency traffic. Since then, temporary repair works have allowed public access through these routes, but, long term repair/replacement strategies are still being considered by the managing authorities. From the perspective of life safety we can consider the overall performance satisfactory. However, based on the observed undesirable sub-system performance of the damaged bridges further investigation into possible improvements of the current design philosophy is warranted. Some discussion points are herein listed and proposed as potential further research areas:

1. Collapse Prevention Limit State: Bar buckling and fracture. Can we confidently predict when buckling and fracture will occur? Also, is it acceptable to have either buckling or fracture occurring at the collapse limit state?
2. At the Awatere River Bridge, cracking and spalling occurred where previous cracking and spalling had been repaired after the Seddon earthquake. Given a further and possibly stronger aftershock, what is the residual fatigue life of the reinforcement?
3. Wall pier bridges and multi-pier bents appear to have sustained less residual drift than some of their mono-pile pier counterparts. Should the design philosophy of mono-pile piers be rethought on the basis that the piers and foundations have low structural redundancy in case of failure?
4. Very high vertical accelerations occurred near the epicentre during the Kaikōura Earthquake leading to deck unseating and displacement of bearing pads on a number of simply supported deck bridges. Should the way elastomeric bearings are attached to bridges be rethought in order to prevent separation of the bearings from the deck? And more importantly is the bearing shear-axial interaction properly captured within the Bridge Manual recommendations?

5. Should the substructure repair strategy for bridges be given more consideration during the conceptual phase of design? This question arises from the observation that the ease of reparability/strengthening depends largely on the choice of the type of substructure (pier and foundation).
6. Many services (especially pipes) were damaged across the abutment-deck joint. This was due to a lack of consideration of displacement compatibility between the bridge and the utility lines. Should there be a change or improvement in the way utility providers work with bridge designers in order to capture the interaction between the bridge and utilities?

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