The New Zealand Concrete Industry
Conference2012
Claudelands, Hamilton 11-13 October

# LESSONS LEARNT FROM CANTERBURY EARTHQUAKES: DAMAGE ASSESSMENT AND NUMERICAL ANALYSIS OF CONCRETE BRIDGES

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## ABSTRACT

In less than six months the city of Christchurch experienced two major earthquakes on September 4, 2010 and February 22, 2011 followed by severe and frequent aftershocks; the former released a magnitude Mw 7.1 earthquake 30-40km away from the Central Business District (CBD); the latter event, a magnitude Mw 6.3, was less than 10km from the CBD. No major bridge collapses were registered; however being Canterbury soils susceptible to liquefaction and lateral spreading most of the traffic disruption and closure of bridges was caused by extensive damage to the approaches and foundation settlements.

Though the majority of damage observed was due to liquefaction and lateral spreading of the river banks, key bridges located in non liquefiable sites and strategic arterial routes, such as Moorhouse Ave Overpass, Horotane, Port Hills Overbridge suffered extensive damage compromising the functionality of Christchurch traffic network.

The present work aims to give a detailed overview of the damage observations referring to the data collected in a database of 800 bridges from various bridge teams from varying organizations. Moreover, for the three above mentioned bridges, detailed non linear static and dynamic analyses supporting the damage inspection results are herein presented with the intent to indentify the key parameters affecting their overall seismic vulnerability. The results from the inspections and detailed numerical analyses highlight unexpected design issues that are not properly detailed in New Zealand standards and need to be further reviewed.

## INTRODUCTION: SEISMIC DEMAND

In less than six months, two important earthquakes, occurring on September 4, 2010 and February 22, 2011, struck the city of Christchurch, New Zealand. The $M_w$ 6.2 February 22, 2011 Christchurch earthquake had an epicentre less than 10 km from the Christchurch CBD between Lyttelton and the South Eastern edge of the city. The close proximity and shallow depth of this event resulted in higher intensity shaking in Christchurch with respect to the Darfield event in September 2010 [1]. Further aftershocks occurred during the following months, with two of the strongest, the Mw 6.0 on June 13, 2011, and on December 23, 2011 with an epicentre again on the South Eastern edge of the city [2]. Horizontal PGAs were in the range of 0.37-0.51g in the Christchurch CBD, while vertical PGAs reached up to 2.1g (Figure 1a).

This shaking level combined with the soil characteristics of the region caused extensive liquefaction and lateral spreading, especially close to the river-banks [3].

Most of Central and Eastern Christchurch area was identified as having high liquefaction susceptibility, with most of this area affected by some level of liquefaction following the Christchurch earthquake [4]. The damaged bridges were located along the Avon River, coinciding with the zone of moderate-severe liquefaction (Figure 1b).

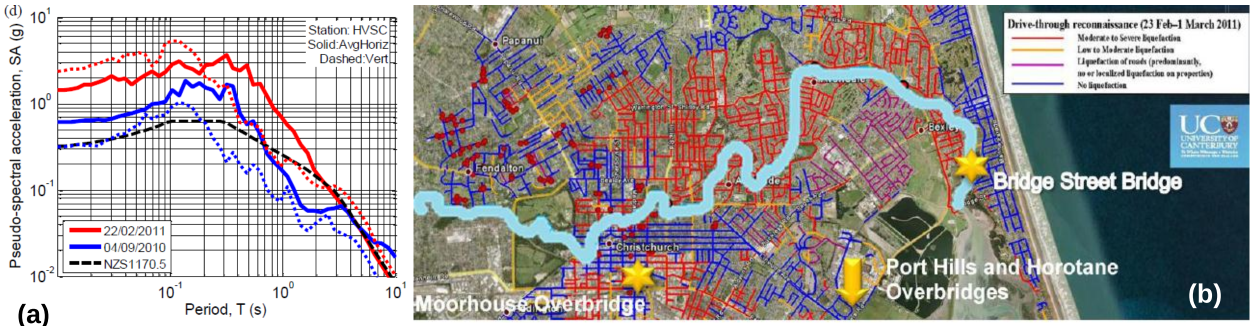


**Figure 1. (a) Extreme ground motions observed Heathcote Valley (HVSC) in terms of pseudo-acceleration response spectra; (b) Map of central and eastern Christchurch indicating bridge locations, strong motion station locations and the region of severe liquefaction damage following the Christchurch earthquake**

The paper aims to give a general overview of the damage sustained by bridges in Christchurch. The first part presents a statistic analysis to quantify the overall damage and to analyse trends leading to the damage of bridges after February 22. This was carried out according to the field observation data from the Bridge Database of the University of Canterbury. In the second part of the paper, a description of the most important effects of the Christchurch earthquake is provided with the support of case studies.

## POST-EARTHQUAKE BRIDGE DAMAGE

### University of Canterbury Database

Following the earthquake, all bridges of the city were inspected by the road network consultants and by researchers from the University of Canterbury. The information was then collected in a database, coordinated by the University of Canterbury. The tool aims to offer an unbiased method for assessing the overall performance of bridges in Canterbury.

The database covers all the bridges in the region bordered by the Ashley River in the North, by the Rakaia River in the South, and by the Castle Hill Area in the West, as this area included the strongest shaking experienced (Modified Mercalli Intensity, MMI ≥ VI) and it encompassed the majority of the damaged bridges after the 4th September 2010 Darfield Earthquake (more than 800). The February 22, 2011 earthquake mainly struck the city of Christchurch, hence the boundaries were narrowed and only 223 bridges of the stock were updated in the database.

Besides the gathering and the classification of the damage, discussed in more detail in the next section, the database provides for each bridge also a series of general information such as construction period, location, geometry, structural typology etc. They were collected from the five authorities who own and maintain the bridge stock: New Zealand Transport Agency (NZTA), ONTRACK (a.k.a. KiwiRail), the Christchurch City Council, OPUS, the Waimakariri District Council, and the Selwyn District Council.

**Damage quantifying procedure**

In order to provide a more comprehensive and detailed damage quantification, the assessment was carried out by looking at bridge components or elements (deck, bearings, abutments, piers, foundation, approaches, services). Figure 2 shows some examples of the main damaged elements of South Brighton Bridge (-43.5252, 172.7241) (Figure 2e) after the 22nd February earthquake.

As shown in Figure 2a, one commonly observed effect of lateral spreading was backward rotation of abutments combined with the inward horizontal movement toward the riverbanks. The latter was sometimes close to 1m and caused bridge superstructure pounding. The deck also acted as a stiff strut while the foundations underwent forced rotations or movements in the direction of lateral ground flow. The lateral spreading of river-banks caused high demands also in the connection within the deck (Figure 2b) and the abutment piles, causing the formation of plastic hinges in the latter (Figure 2c), which in some cases were exposed.

Pier damage was caused either by ground shaking with a combination of vertical and horizontal PGAs, or by lateral spreading. Lateral spreading caused a lateral force induced by the ground movement applied to the pier/column foundations. This force was not predicted in design and therefore the internal actions generated by it caused cracking of pier.

Bridges constructed in 1950s-60s were very robust and sturdy. Therefore the above-mentioned forces didn't bring the bridge to collapse. Figure 2d shows an example of typical deck damage which was caused by pounding against the abutment. In general, the damage was most evident between the bridge deck and the approach. Lateral spreading of river-banks caused significant differential movements; the formation of large vertical offsets at the ends of the bridge, made its access difficult or impossible.

Bridges generally affected the movements of river-banks. In fact, while the lateral spreading crack pattern was usually parallel to the river, the presence of the bridge restrained the lateral spreading movement inducing a sort of "strut effect", resulting in cracks perpendicular to the river as shown in Figure 2f.

Many bridges are "utility links" for the Christchurch lifelines. Many services, such as power, water and waste-water pipes were running along the bridges. Damage of these services was significant. The relative displacements of the bridge superstructure to bridge abutments and foundations, induced by either the ground motions or land-movements caused leaking/breaks of linkages on water and waste-water pipes and distortion of power and telecommunication cables (Figure 2h).

Based on the field observations, the damage quantification procedure was structured by identifying the components and quantifying their associated damage. Nine members/components were considered: deck and superstructure; bearing between deck and abutment or piers; pier; abutment; bridge pavement; approach pavement; approach settlement; services crossing the bridge and the surrounds in the interaction zone with the bridge. The damage for each member was then quantified by a given value which identifies its damage limit state. A total of four damage states (ds) were defined for bridge components: d0 (none), d1 (slight/ minor damage ), d2 (moderate damage) and d3 (extensive or complete damage).

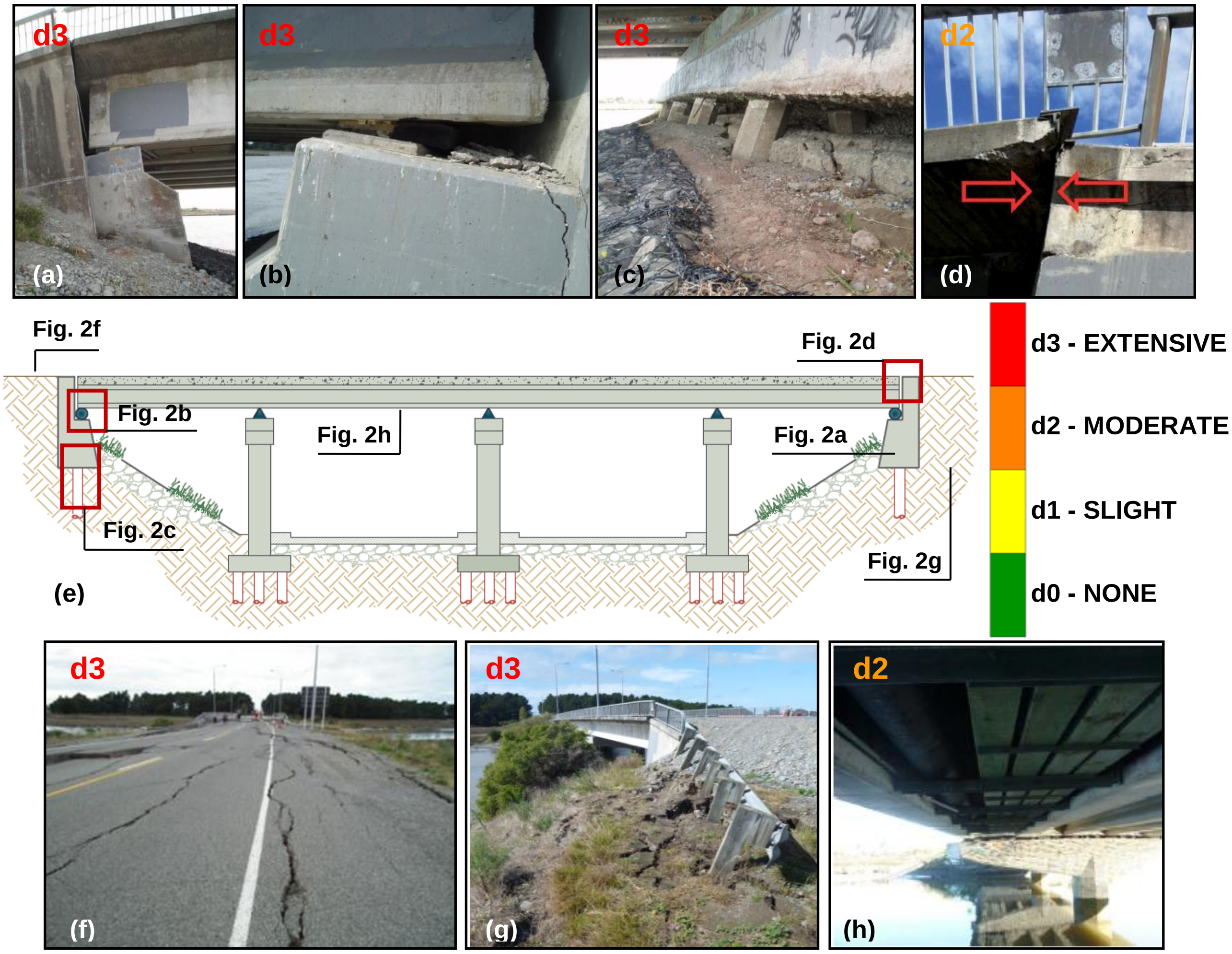


**Figure 2. Damage at South Brighton Bridge with the indication of the level damage according to the database scale: a) Rotation of the abutment; b) Damage at the bearing; c) exposure of the piles with the formation of plastic hinges; d) Pounding of the deck on the abutment; e) general view of the bridge; f) Damage at the approach pavement; g) West approach settlement; h) Damaged pipelines.**

## Field observation data: the results

As already discussed, the database gathers information about damaged components as well as the bridge typological characteristics. Statistics of the bridge stock is shown in Figure 3. Figure 3a shows that more than 50% of the bridges are built in concrete, while timber and steel have relatively equal percentages. Masonry and mixed bridges take up less than 5%. With regards to the categories of use, as expected, most timber and steel bridges are pedestrian with respectively 88% for timber and 67% for steel. Concrete material was instead mostly used for road bridges (81% full cast in place, 68% with precast concrete decking).

The damage index classification is essential for a proper unbiased and consistent overview of the post-earthquake damage. The criterion adopted to obtain a global damage index (GDI) was to sum the severity ratings of the different damage categories. Therefore, for example, a bridge with a damage severity of 2 for the deck, and 3 for the bearings would be given a GDI = 5. In order to rate the bridge, four damage classes were defined: a GDI ranging from 1 to 2 was classified as minor bridge damage, medium damage was set as GDI=3 to 9, and severe total bridge damage was set for a GDI greater than or equal to 10. Two of the damage components (pavement / approaches) previously discussed are not independent; the former is actually a subset of the latter. In order to avoid damage overestimation, in the sum of the damage components for the bridge only the maximum was used to respect their dependence on each other.

Results are shown in Figure 3b. The general bridge performance during the earthquake was satisfactory, with only 4% of bridges sustaining severe damage [5]. In other words, for the majority of bridges in the database, the Christchurch earthquake was a 1/500 year design level event. Considering that the NEHRP Recommended Seismic Provisions (National Earthquake Hazards Reduction Program) recommend that in a maximum credibility event (1/2500 year event), the probability of collapse should be less than 10% for an ordinary structure, the amount of severe structural damage to bridges for these 1/500 year events is appropriate.

Lastly Figure 3c gives a preliminary estimation of damage to the bridge components after February 22nd 2011 event. From the histogram it is evident that of all bridges examined, 70% presented damage to the "non structural parts" of the bridge system. Lateral spreading was identified as the main cause of the damage. Results clearly demonstrate that a new holistic approach, which involves an integrated design between structural and non-structural parts of bridges, is needed.

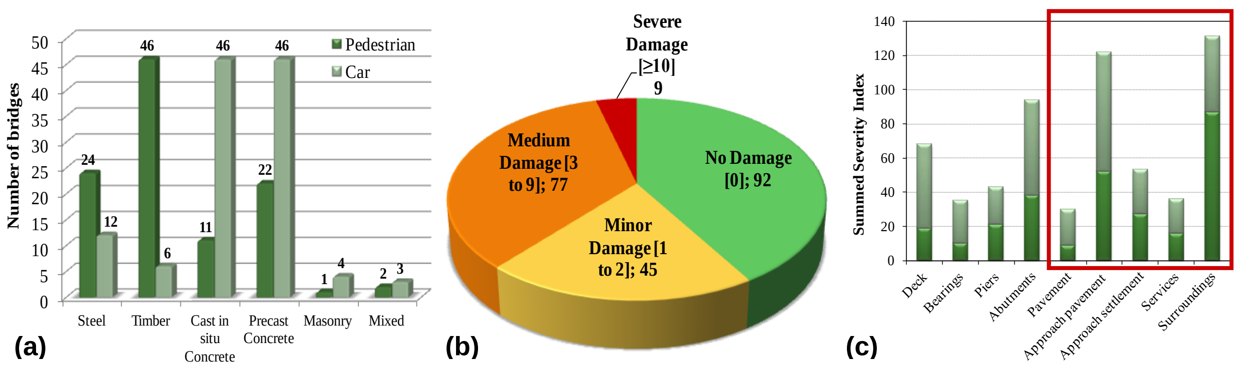


**Figure 3. a) Proportions of the bridge category of use for each structural material type; b) Christchurch Bridges stock grouped on the basis of the damage severity after the February 22 earthquake; c) Summation of the damage severity index for the different categories of damage. With bridges divided into those affected and not affected by lateral spreading.**

## EFFECTS OF THE GROUND SHAKING

### Vertical accelerations effects

During the Mw 6.1 February 22, 2011 Christchurch earthquake, despite the short duration (15-20 seconds), very significant ground motion amplitudes were recorded in both the horizontal and vertical components. The maximum PGA's in the vertical component was 2.21g in the Heathcote Valley (HVSC), recording one of the highest maximum Peak Ground Acceleration in the world experienced close to a city centre. Due to the shallowness of the earthquake, vertical accelerations were generally high especially in the East part of the City ranging from 0.8 to 2.1g. To prove the importance of this parameter, a state highway bridge close to Port Hills (-43.5711, 172.6934), is herein analysed.

The bridge, constructed in 1963, is a dual, six span, simply supported bridge (Figure 4a). The prestressed concrete log type beams are connected by a cast in place concrete slab. The spans vary in length between 9.4 to 12.6 m and the overall length of the wider bridge is 72.4 m. The bridge abutments and the reinforced concrete single stem rectangular piers are cast in place on spread footings.

As part of seismic retrofit programme [1], the overpass bridge has recently been strengthened by fixing fabricated steel shear keys to the underside of the beams at both the abutments and piers to resist longitudinal earthquake loads. Linkage rods were fitted between brackets located on either side of the piers by drilling through the tops of the piers to form a tight linkage between the adjacent spans. The down-stand of the brackets prevents relative movement between the spans and the piers. Linkage at the abutments was provided by rods extending between the brackets and the abutment side facing the soil. New shear keys fixed to the the abutment and pier sides provided resistance to transverse loads. Circular steel shrouds were added at one pier column on each bridge to prevent soil restraint of the pier, which may have cause undesired plastic hinging due to ground motion in the transverse direction.

During the February event, flexural cracking developed in the lower halves of all pier stems of both bridges except those adjacent to the abutments. In fact, despite the increased deck stiffness, the central piers underwent displacements greater than their yielding displacements. Soil gapping at ground level occurred at the faces of most of the pier stems with separation cracks up to 15 mm wide. Spalling occurred at the base of the central pier, and the longitudinal bars buckled at the corners (Figure 4b). The damage increased during the June 13, 2011 aftershock. The nominal 10mm gaps between the new shear keys and the abutment face at the South-East abutment closed up with no clearance on two of the four keys.

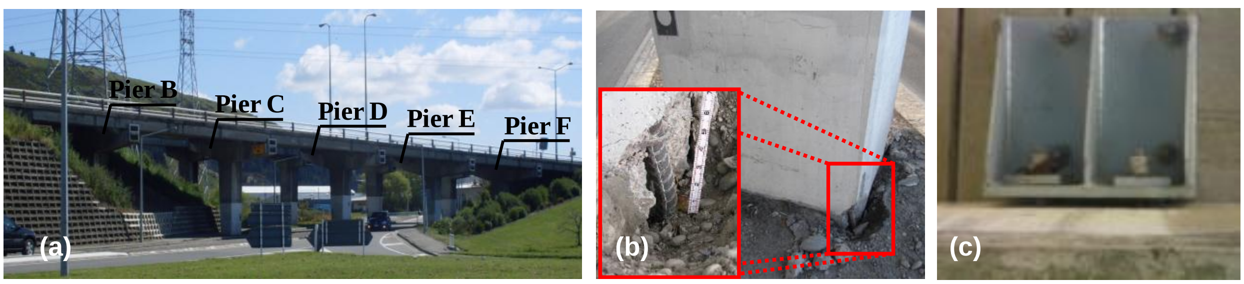


**Figure 4. a) Overall view of Port Hills Road Overpass b) Buckling of reinforcing steel at the base of the piers [Courtesy of OPUS]; c) Elongation of the links between the spans [Courtesy of OPUS].**

The performance of the bridge was also assessed through a numerical model implemented in Ruaumoko3D [6]. Non linear dynamic time history analyses were carried out using the closest recorded accelerograms. The HVSC station seems to be the closest station and representative of the real earthquake demand occurring to the bridge. As shown in Figure 1a, the HVSC spectral acceleration is characterized by a short period ($T < 0.4$s) ground motion with a rapid fall-off in spectral ordinates at longer periods. The trend is similar for both horizontal (solid line) and vertical (dashed line) accelerations. The natural period of the structure was found to be in the range of 0.17 - 0.27s and the vertical accelerations acting on the central pier resulted in the axial force – time curves shown in Figure 5a. This led to a significant variation of the axial force in the columns, which caused a curvature ductility reduction of the piers for positive increments, as shown in Figure 5b. When the axial load variation sums to the static load there is an increment of 39 % in the moment capacity but a reduction of 49% curvature ductility respect to the situation with no axial variation effect. Conversely for the lowest total axial load ($N$-$\Delta N$), the moment capacity reduced by 28% while the ductility increased by 94%. The capacity reduction caused the longitudinal bars buckling. According to numerical analyses, this occurred when the deck displacement reached 3.30 cm.

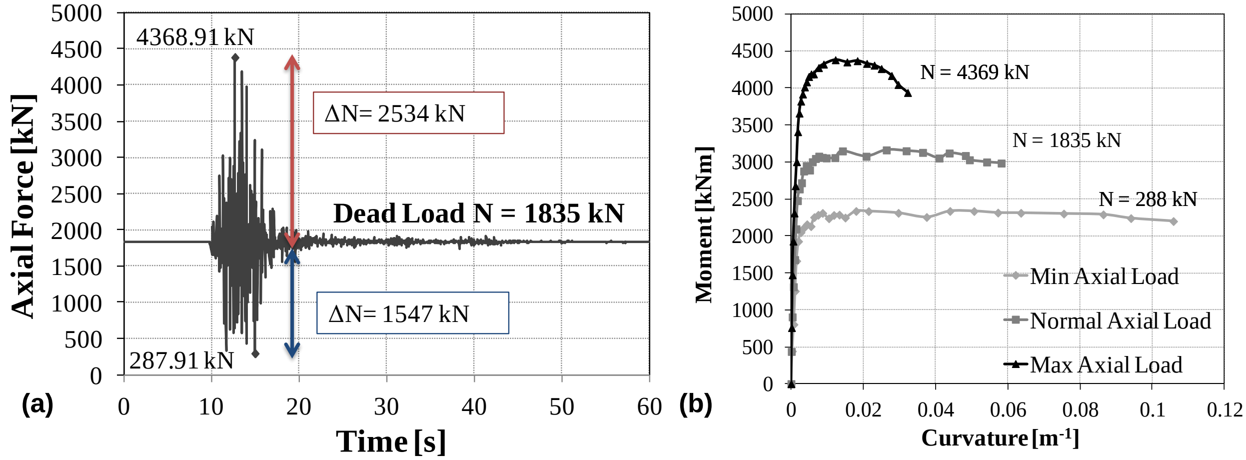


**Figure 5. (a) Time history of the axial force loading the central pier (Pier D) when subjected to HVSC records. (b) Evolution of the moment-curvature with varying axial force.**

## Horizontal accelerations and bridge irregularities

As for the horizontal accelerations, close to the Avon River (East part of the City), PGAs ranging from 0.19g to 0.63g were recorded, while in Christchurch CBD, they were in the range of 0.37-0.51g. The horizontal components were less exceptional then the vertical ones, but certainly still significant, especially if compared to the Darfield event (4th September 2010) where PGA's around Christchurch typically ranged between 0.2 & 0.35g [7]. However for natural periods T < 0.4s, typical of Christchurch bridges, the earthquake demands did not exceed the NZS1170 design spectral values [8]. The structures in the CBD had a good dynamic response, even if the conclusion needs to be dampened a little since the bridge was built according to different standards. The regularity, robustness and the short spans certainly contributed to this satisfactory performance.

However, some bridge irregularities given by lack of proper design detailing have been seen and this caused unexpected and unpredicted damage to the structure. An example is Moorhouse Ave Overbridge, (-43.5399, 172.6367) (Figure 4a), an eleven span reinforced concrete structure and one of the four avenues that encase the CBD of Christchurch City, allowing traffic to flow around the CBD. Built in 1964, the reinforced concrete T-beam superstructure is supported by two column bents. The bridge is founded on 406 mm diameter octagonal reinforced concrete piles. The structure was constructed in three separate sections, linked with expansion joints. The performance during the February event was unsatisfactory: although the results of the preliminary spectral analysis are good, the insertion of steel rod linkages in the deck only at the expansion joint between the west and the central part of the bridge caused irregularity in structure and the mechanism of damage shown in Figure 6b.

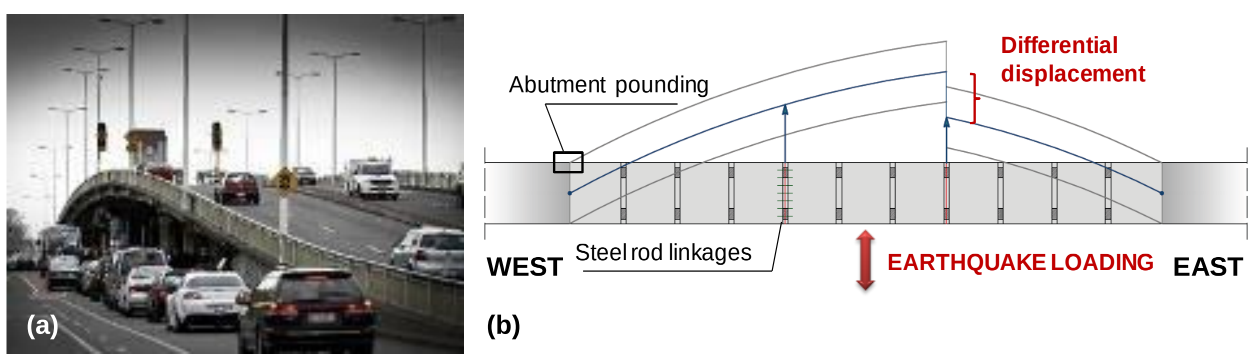


**Figure 6. a) Overall view of the bridge; b) Plan on bridge showing qualitative displacement profile under transverse loading.**

The pier at the East expansion joint suffered extensive displacement demand. The slenderness of the pier affected the vertical load carrying capacity of the structure along with the lateral capacity. The columns had also widely spaced transverse reinforcement, making the structure susceptible to a brittle failure mechanism (Figure 7b). Observations after the Christchurch event indicated that the damaged columns had started to buckle putting the central span at risk of collapse (Figure 7c-d). Due to the higher displacement demand in the West-Central part of the bridge, the deck pounded against South-West abutment of the bridge causing extensive spalling and bar bucking.

Figure 7a shows the results of the time history analysis carried out on the numerical models with the CCCC records of the Darfield and Christchurch earthquake. The blue line represents the maximum displacement profile of the structure during the February event; it is clearly highlighted the asymmetry in the response of the bridge with significant different displacement demands in the pier 7.

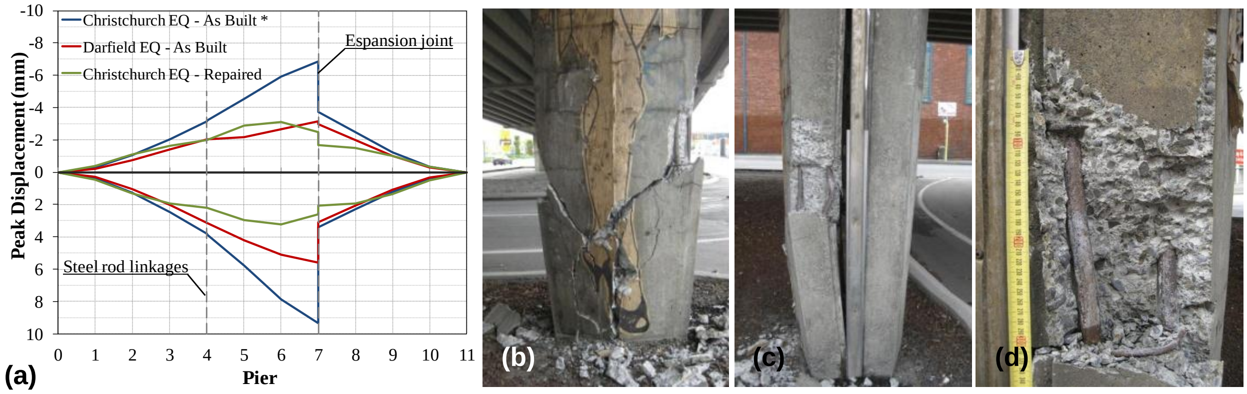


**Figure 7. a) Displacement profile of each model; South column: (b) West face (c) South face; d) Exposure of the steel bars at the North face of the north column.**

## EFFECTS OF LIQUEFACTION, LATERAL SPREADING AND SLOPE FAILURE

Most of the bridge damage was mainly due to liquefaction induced lateral spreading throughout the Eastern part of the City and in the rural areas in the surroundings. The effects of February event were mainly localized in the East part of the city, while the Darfield earthquake caused wide-spread , but less significant land-damage. The high intensity level of the ground motions resulted in large areas of severe liquefaction damage [9,10]. The bridges along both the Avon and Heathcote Rivers suffered from varying levels of damage due to lateral spreading, depending on the ground conditions and the ground shaking level. The effects of the lateral spreading were more significant in precast bridges, such as ANZAC bridge (Figure 8a), where the foundations underwent forced rotations or movements in the direction of the lateral ground flow (Figure 8b). Preliminary analyses have been carried out to determine lateral loading capacity of the piled abutment and the location of potential plastic hinges in the piles [10,11].

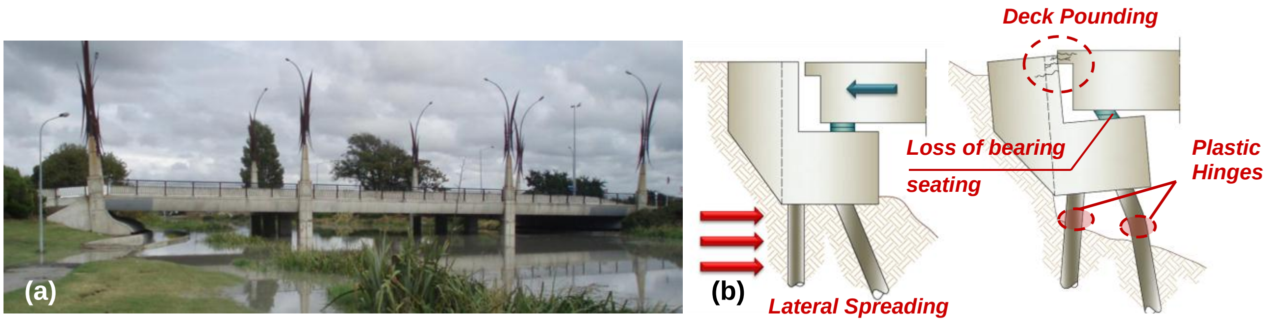


**Figure 8. a) Overall view of ANZAC Bridge; b) Sketch of the mechanism of the lateral spreading.**

Slope failures of the embankments were recorded in the Port Hills area, close to the epicenter of the February event. Horotane Valley Overpass (-43.5725, 172.6947) (Figure 9a) suffered extensive damage and there is still speed restrictions due to loss of vertical alignment.

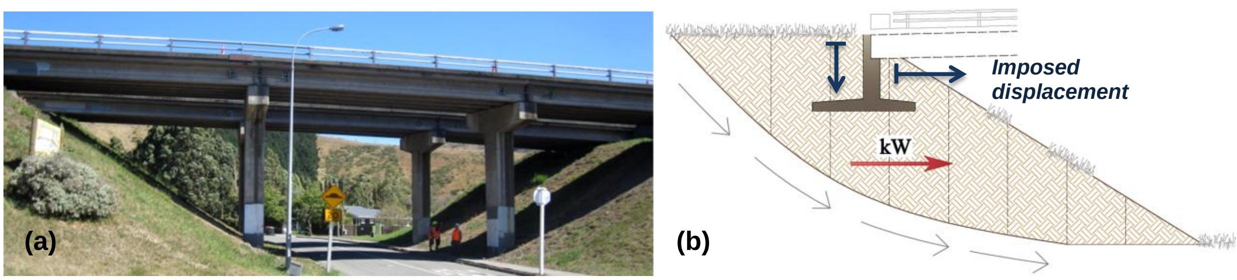


**Figure 9. a) Overall view of Horotane Valley Overpass; b) Cracking from down-slope movement at abutment at Lyttelton end; c) Cracking in abutment at Lyttelton end.**

The bridge, constructed in 1963, consists of twin bridges each carrying two lanes of SH 74 across Horotane Valley Road (Figure 9a). Both bridges are similar, with three simply supported spans and prestressed concrete beams supporting a reinforced concrete deck. The spans are 13.9 m and 12.5 m for the end and central spans respectively. The bridge abutments and the single stem rectangular piers are founded on spread footings. The spans are well linked to the abutments and piers by both holding down dowels and linkage rods.

The bridge has recently been strengthened by fitting steel shear keys at the abutments, primarily to resist transverse loads. Each of the nine brackets at each abutment (single abutment structure for both bridges) is fitted with a 30 mm diameter bolt into the bottom of the beams. This provides additional longitudinal restraint in addition to that provided by the original linkage. Linkage rods were also added between the outer beams at each pier cap. These were designed to improve the deck diaphragm action under transverse loading and avoid unseating.

The structure is very stiff; this was also confirmed by the numerical analyses as the natural period was found to be in the range of 0.085 and 0.092 s. The stiffness positively influenced the performance of the bridge, which did not suffer any damage due directly to the ground shaking. Nevertheless the strong motion caused slope failure of both the embankments, which resulted in some damage of the structure.

In fact following the Christchurch earthquake, the east end of the No 2 Bridge displaced horizontally about 100 mm in a southwards direction. This resulted in severe vertical cracking at the junction between the abutment seating and the abutment wall between the two bridges. There was minor spalling at the ends of the beams where they were seated on the abutments. All four abutments appeared to have moved forward by up to 20 mm. This movement caused severe shear cracking in the back-wall and shearing of two of the bolts on the new linkage brackets (loaded in shear) at the west abutment of the No 1 Bridge (Figure 10a). Bolts on the new linkage brackets also sheared on both abutments of the No 2 Bridge. The west abutments had settled by about 60 mm. This was particularly visible on the south side of the No 2 Bridge. Surface sliding of soil was evident under the west abutments and wide cracks and separation gaps between the soil and abutments were evident at the east abutments indicating significant down-slope movements (Figure 10b).

A simplified numerical model based on Newmark methodology [12], which provides an index of permanent deformations, has been developed [13]. Dynamic non linear time history analyses which includes the activation of the slope failure mechanism have been carried out. Figure 10c shows the results of the analyses (shear force vs time), with the indication of the breaking point of the retrofit bolts. Their failure occurs after 10 seconds in correspondence with the activation of the slope movement.

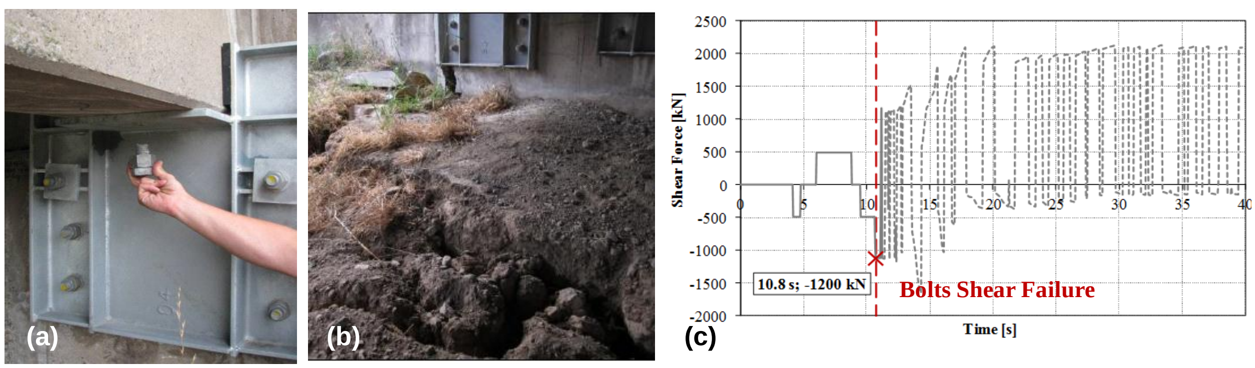


**Figure 10. a) Cracking from down-slope movement at abutment at Lyttelton end; b) Time History of the Shear Force acting on retrofit bolts, with the indication of the breaking point; c) Sheared bolt at the abutment retrofit [Photo by J. Allen].**

## CONCLUSIONS

The overall performance of bridges after the February 22, 2011 earthquake was satisfactory. This was also confirmed by the statistical results obtained using the Bridge Database of the University of Canterbury: in fact only the 4%, of the entire stock was considered severely damaged. Results also confirmed the lateral spreading as one of the main causes of the damage. This phenomenon was damaging especially for precast bridges. On the other hand, the robustness of Christchurch City Council road bridges built in the 1940s and 1950s without any seismic design criteria certainly helped to sustain earthquake loadings comparable with or higher than the current design levels.

The 70% of the Canterbury bridge stock was damaged to the non-structural parts (pipes, approaches, road). There is the need for preserving not only the structure, but also the "accessories". The interruption of their functionality caused extensive business disruptions to the Christchurch community. Few pipe leaking or breaking as well as differential vertical displacement at the bridge supports caused the closure of roads, water/waste water lines impacting on the whole lifeline systems of suburbs, such Dallington, Avonside etc. Improvements in this sense need to be done as New Zealand Standards as well overseas codes are deficient in terms of design integrated approach.

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