Assessment of bridges in non liquefiable soils following Canterbury earthquakes: comparison between observations and numerical analyses

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SUMMARY:
On February 22nd, 2011 a M_w6.2 Earthquake occurred with an epicentre near the town of Lyttelton, 10 km South of the Christchurch Central Business District (CBD), New Zealand. Though the majority of damage observed was due to liquefaction and lateral spreading of the river banks, examples of significant bridge damage on non-liquefiable sites occurred as well. The overall damage suffered by the bridge stock of the Canterbury Region, following the last earthquake, has been documented and collected into a Bridge Database, which is herein analysed.
The second part of the paper focuses on the seismic performance of the three key concrete bridges not subjected to liquefaction: Port Hills and Horotane Overbridges, and Moorhouse Overpass. The assessment involves, site investigations, numerical modelling including quasi-static and dynamic analyses. The models included features such as shear and axial bending interaction and the soil interaction with the structure to properly capture the damage observed during their reconnaissance.

Keywords: Lyttelton Earthquake, vertical accelerations, concrete bridges, damage assessment, time history analyses

1. INTRODUCTION

In less than six months the city of Christchurch, New Zealand, experienced two major earthquakes, occurring on September 4, 2010 and February 22, 2011. During the last event, although the magnitude was only 6.2, the close proximity to the city centre, 10 km, and the shallow dept of the origin, 5 km, caused high intensity shaking in Christchurch with PGA up to 1.4g. In the CBD, ground motions were characterized by large vertical accelerations as well (up to 2.1g), resulting from a reverse-thrust faulting mechanism. This shaking level combined with the soils characteristics of the region caused liquefaction and lateral spreading of the river banks in Christchurch (Cubrinovsky *et al.* 2011) with the majority of bridge damaging abutments, piled foundations and approaches (Wotherspoon *et al.* 2011, Palermo *et al.* 2011).

The typical pre60s monolithic construction (integral bridges) and the very high robustness of Christchurch bridges allow them to sustaining the axial demands placed on the structure due to lateral spreading, even though they were not specifically designed for these seismic actions. In fact although the damage threshold level was much lower than the estimated bridge response accelerations in the earthquake, the global performance of bridges was satisfactory, with only few bridges with visible structural damage (Palermo *et al.* 2011).

These observations are confirmed by the statistical analysis carried out using a database which has been developed at University of Canterbury. The first part of the paper aims to provide a unbiased picture of the situation after the last seismic events. For sake of brevity, the statistical analysis will be brief and more details can be found in (Brando 2012). Although most of the bridge damage was a result of liquefaction and lateral spreading of the river banks, the paper limits the detailed analyses to bridges in non liquefiable soils. Another joint paper (Wotherspoon & Palermo 2012) will present analyses on bridges subjected to liquefaction and lateral spreading. Three bridges, have been selected:

Port Hills and Horotane Valley Overbridges and Moorhouse Ave Overpass. These bridges are key links for the arterial road routes of Christchurch city. After, a brief summary on damage observations, numerical modelling which includes features such as shear and axial bending interaction (Priestley et al. 1996, Bresler 1960) and slope failure are herein presented. Results from non Linear time history analyses adopting Christchurch record sequence are compared with the damage assessed during the bridge inspections.

2. SEISMIC DEMAND

The M_w 6.2 February 22, 2011 Christchurch earthquake had an epicentre less than 10km 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 relative to the Darfield event in September 2010 (Palermo *et al*. 2010). Further aftershocks occurred during the following months, with one of the strongest, the Mw 6.0 on June 13, 2011, with an epicentre again on the South Eastern edge of the city (GNS Science 2011).

Horizontal PGAs were in the range of 0.37-0.51g in the Christchurch CBD. Significant vertical accelerations were also registered. Strong motion records indicated that most of the bridges within 10 km of the Christchurch earthquake epicentre were subjected to 0.25-1.4g horizontal PGAs. In the Port Hills area a horizontal PGA of 1.41g was recorded near the epicentre at the Heathcote Valley Primary School (HVSC) strong motion station. Figure 1a shows that the short period spectral accelerations were very high at several stations (PRPC, HVSC) close to the fault rupture (Geonet 2011). Acceleration response spectra of typical sites from the Christchurch event are compared with the New Zealand Design Spectra (NZS 1170.5:2004) for site soil class D, 500 year return period.

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 (Wotherspoon *et al*. 2011). The damaged bridges were located along the Avon River, coinciding with the zone of moderate-severe liquefaction. Although bridges crossing the Heathcote River were closer to the fault rupture, they suffered less extensive damage being the soil conditions of region not liquefaction-prone. (Figure 1b). Exception is for the Ferrymead Bridge (-43.5584, 172.7086), located at the mouth of the Heathcote River, which has been closed several days and had traffic weight restrictions. All other bridges were either undamaged or suffered only minor damage.

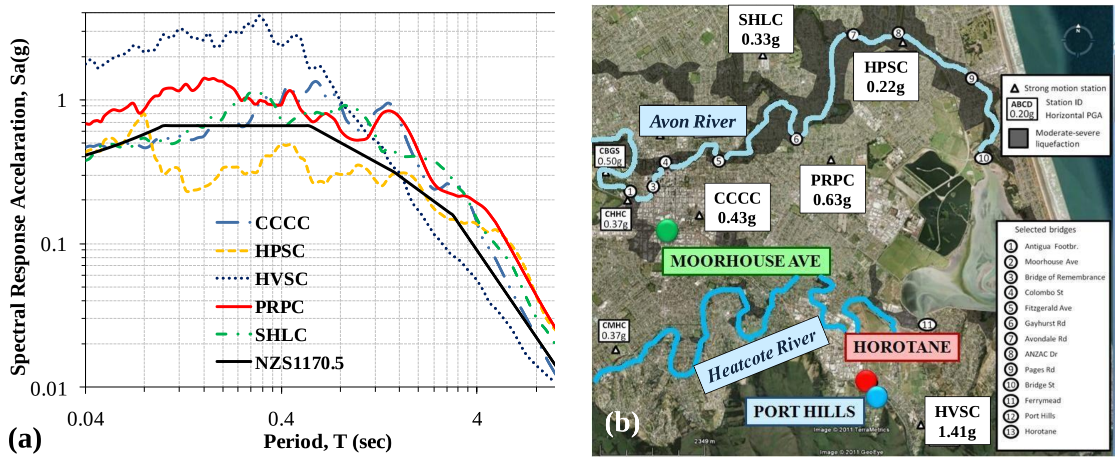

Figure 1. a) Response spectra of the geometric mean of the horizontal accelerations at strong motion station in Central and Eastern Christchurch compared to NZS1170.5 design response spectrum for Christchurch, site subsoil class D for a 500 year return period. Four letter symbols represent different strong motion stations. b) Overview of Christchurch and the surrounding region, with locations of the case studies bridges and strong motion stations.

3. FIELD OBSERVATION DATA: RESULTS FROM THE DATABASE

In order to offer an unbiased method of assessing the performance and cause of damage to bridges in Canterbury after the two last earthquakes, a database which collected all observations data and key parameters of bridges has been developed at University of Canterbury. In general the database covers all the bridges in the region bordered by the Ashley River in the North (-43.2770, 172.6293), by the Rakaia River in the South (-43.5879, 171.74059), and by the Castle Hill Area in the West (-43.2053, 171.7179). This area includes the strongest shaking experienced (MMI ≥ VI) and it encompasses the majority of the damaged bridges after the 4th September 2010 Darfield Earthquake. While Darfield event required to input data for 800 bridges, the 22nd February 2011 Lyttelton Earthquake, struck mainly the Christchurch area, so that only 223 bridges were inspected and updated in the Database. The boundaries are so narrowed and reach out the edge of the city (Figure 1b).

In order to process the analyses of data, it was necessary to find a criterion to move from the qualitative to the quantitative assessment of the damage. To make the assessment easier and at the same time more precise, bridge damage was split per components: deck and superstructure; bearing between deck and abutment or piers; pier; abutment; bridge pavement; pavement; approaches services crossing the bridge. Figure 2a shows an example of damage at the abutment of Bridge St. Bridge (43.5252, 172.7241). The damage in each category was given in a numerical value based on its severity. A total of four damage states (ds) were defined for bridge components: 0 (none), 1 (slight/ minor damage), 2 (moderate damage) and 3 (extensive or complete damage).

First of all, the bridge stock was analysed in terms of mechanical and geometrical characteristics. The percentage of the total bridge stock for structural material type is given in Figure 2b: over half the bridges in the surveyed region were concrete with the 30.5% precast and the 25.6% cast in place; timber and steel surprisingly have relatively equal proportions, while masonry and mixed bridges cover a very small percentage.

A first global damage assessment caused by the Christchurch Earthquake of February 22nd can be found in Figure 2c. To summarize the total damage in the assessed region a representative index for the whole bridge was proposed. It consisted of the sum of the severity damage ratings of the different damage categories. To place the bridge in one of the four classes of damage a summed damage of 1 to 2 was classified as minor bridge damage, medium damage was set as 3 to 9, and severe total bridge damage was set as greater than or equal to 10. Pavement and surround/approach damage, although it was helpful to have them distinguished, are quite dependent. Therefore, in order not to overestimate the damage of a particular bridge, in the sum of the total damage only the maximum of these two categories was used to respect their dependence on each other.

The results confirmed that general performance of the bridges during the earthquake was good, with only the 4% of bridges with severe damage observed, more details can be found in (Brando *et al.* 2012). For the majority of bridges having a period in the range of 0.2-0.8 seconds Canterbury earthquake events and in particular 22nd February were similar and greater than 1/500 year design level event (Palermo *et al*. 2011). 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 can be considered acceptable. However, although the life safety design level was satisfactory, the traffic disruption generated by the damage to pavement, approaches and services was very high and therefore compromising the functionality of Christchurch network for several months.

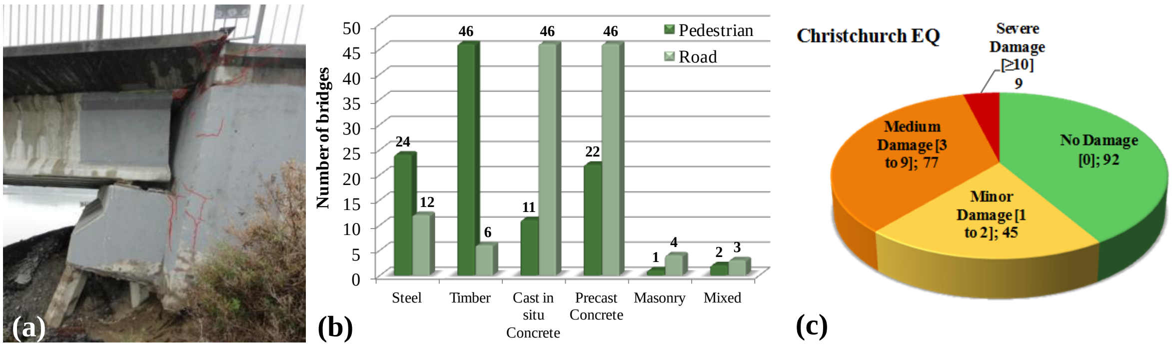

Figure 2. a) Example of damage to the abutment: rotation at Bridge Street Bridge; b) Proportions of the bridge category of use for each structural material type. c) Christchurch Bridges stock grouped on the basis of the damage severity after the February 22 earthquake

4. CASE STUDIES

This section presents a detailed seismic analysis of the only three road bridges in which the liquefaction was not the cause of the damage: Port Hills and Horotane Valley Overbridges and Moorhouse Ave Overpass. Numerical modeling uses lumped plasticity elements and is developed through Ruaumoko 3D (Carr 2008). The column are modeled using a giberson model which take into account axial load effect (Bresler 1960) while elements in the deck are elastic. The cyclic behavior of plastic hinges bridge piers is modeled through Takeda hysteresis type (Otani 1974). The non linear dynamic time history analyses are carried out using the registrations recorded during the Canterbury earthquakes in the stations close to the bridges (Geonet 2011).

4.1. Port Hills Overbridge

4.1.1. Description of the structure and earthquake damage

The Port Hills Overbridge (-43.5711, 172.6934), constructed in 1963, is a dual six simply supported span bridge (Figure 3a). The prestressed concrete log type beams are connected by a cast-in-situ 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 situ too and founded on shallow spread footings.

As part of seismic retrofit programme (Chapman *et al.* 2005), the Overpass bridges have 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 adjacent spans. The down-stand of the brackets prevents relative movement between the spans and the piers. New shear keys fixed to the faces of the abutments and piers provided resistance to transverse loads. Circular steel shrouds were added at one pier column on each bridge to prevent soil restraint to the pier and maintain the bridge regularity. This may have caused undesired plastic hinging due to ground motion in the transverse direction with higher damage in the central pier (pier D) due to the higher displacement demand induced by the earthquake.

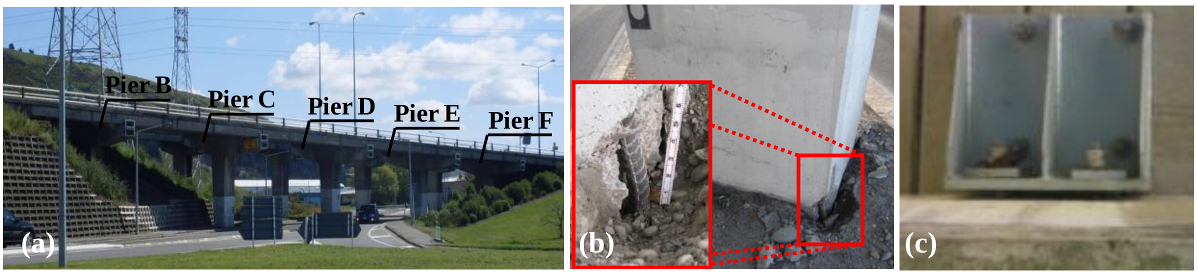

Figure 3. a) Overall view of Port Hills Road Overpass b) Buckling of reinforcing steel at the base of the piers [Courtesy of OPUS]; c) Elongation of the links between the spans [Courtesy of OPUS].

In the Christchurch earthquake, flexural cracking developed in the lower halves of all pier stems of both bridges except those adjacent to the abutments. In fact, despite the presence of the linkage rods increased the deck stiffness, this was not enough to control the higher displacement demands occurring in the central piers. 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 started to buckle (Figure 3b). 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.

Slope failure at the abutment slopes under the bridge resulted in wide cracks in the soil and at the interface between the soil and abutment face under the South-East abutments. There was also minor settlement and displacement of the approach pavement and back-fill at the abutments.

4.1.2 Performance assessment

In order to assess the performance of the bridge during the February 22, 2011 earthquake, time history analyses were carried out using Ruaumoko3D (Carr 2008) imposing the closest recorded accelerograms to the structure. The piers are fully fixed at the base while at the top they are free to rotate around the longitudinal axis of the bridge. This assumption seemed to be consistent with the damage observed in the piers. Because of the presence of linkage rods, the deck was assumed continuous. Two different abutment-deck restraint conditions were considered, a bridge deck fully fixed or free to rotate around the vertical axis. The second assumption has been adopted in order to model the observed relative movement between the deck and the approaches which resulted in some cracks transversal to the bridge axis. Records used for the analyses were the accelerations measured at the closest SMA Heathcote Valley Primary School (HVSC) station, Christchurch Cashmere High School (CMHS) and Catholic Cathedral College (CCCC). The design acceleration spectrum (NZS 1170.5:2004) was compared to the spectral accelerations of the recorded registrations in the range 0.17-0.27 s (natural period of the structure). While the CCCC and CMHS records match quite well the design spectrum, the spectral accelerations from HVSC are much greater than the design ones, reaching a value of 1.2g. Moreover, as already described in (Palermo *et al.* 2011), this event caused exceptional vertical accelerations with values up to 2.1g in HVSC station.

This led to a significant variation of the axial force on the columns, which caused ductility reduction of the piers as shown in Figure 4a for the central pier (Pier D). When the axial load variation (HVSC record) caused by the earthquake sums to the static load there is an increment of 38.60 % in the moment capacity but a reduction of 49.24% curvature ductility respect to the situation with no axial variation effect. Vice versa for lower total axial load (N-ΔN) the moment capacity reduces of 28.09% while the ductility increases of 93.69%.

Results show that the bridge performed well remaining in the elastic range when subjected to CCCC and CMHS records (with vertical acceleration of 0.8g). This inconsistency can be due to the distance of the stations from the bridge and the epicentre; even if the soil conditions were assumed to be similar, the PGA recorded at this station is expected to be smaller than that felt at the bridge site. A good correspondence between the numerical analyses and the reality is found subjecting the structure to the HVSC records. However, the "real" axial load variation in the bridge was probably smaller since accelerations of HVSC incorporate seismic amplification effects. Figure 4b shows that the displacement demand for the model with fully fixed abutment (FFA) is lower than the displacement at the yield of the base section of the piers. On the contrary, the model with the superstructure pinned at the abutment locations develops plastic hinges at the bottom, with top displacements greater than the yielding ones. The real response of the bridge is hence likely to have been between the ones of the two "limit cases" model. In fact, since the bridge is prefabricated with post-installed mechanical connections, the fully fixed model overestimates the stiffness of the retrofit linkages between the abutment and the deck.

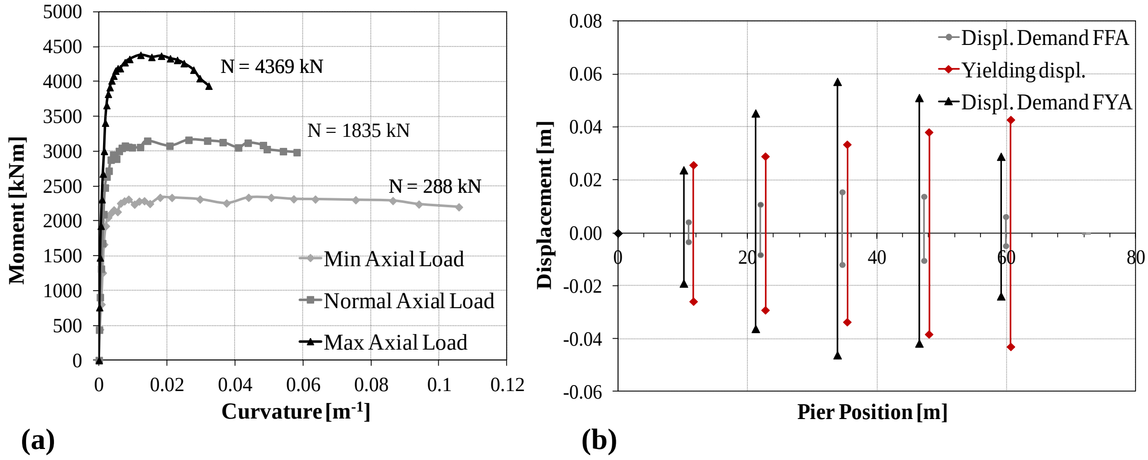

Figure 4. a) Evolution of the moment-curvature with varying axial force. b) Comparison between the displacements of the top of the columns subjected to HVSC records.

4.2. Horotane Valley Overbridge

4.2.1. Description of the structure and earthquake damage

The Horotane Valley Overpass (-43.5725, 172.6947), constructed in 1963, consists of twin bridges each carrying two lanes of SH 74 across Horotane Valley Road (Figure 5a). The Overpass is located about 2 km west of the Christchurch portal to the Lyttelton Road tunnel. Both bridges are similar except that the No 2 Bridge carrying the west bound lanes widens towards its west end to provide additional width for an off-ramp. Each bridge has three simply supported spans with 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 Overpass bridges have 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. Additional linkage rods were added between the outer beams at each pier. These were designed to improve the deck diaphragm action under transverse loading and avoid unseating.

Following the Christchurch earthquake, fine horizontal cracking was observed on the lower half of all four piers, located between about 500 to 2600 mm above ground level. 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 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 backwall 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 5b). 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 5c).

Figure 5. a) Overall view of Port Hills Road Overpass, looking west; b) Sheared bolt at the abutment retrofit [Photo by J. Allen]; c) Slope failure at the abutments [Photo by J. Wood].

4.2.2. Performance assessment

A similar model to Port Hills bridge has been developed for this bridge too. For Horotane Overpass, consistently with the damage observed, the results confirmed that the ground shaking was not the primarily cause of the bridge damage. This is probably due to its high stiffness as the natural period falls in the range of 0.085-0.092 s. Therefore the slope failure has to be identified as the main cause of the damage and a simplified model based on Newmark methodology has been developed (Richards & Elms 1977). An estimate of the horizontal abutments movement to be expected is obtained using the simplified method of Newmark (Newmark 1965) which provides an index of permanent deformations. According to it, sliding of the abutment relative to the soil is initiated when the soil acceleration reaches the limiting value k_y (yield coefficient): every time the ground acceleration exceeds the yield value k_y, permanent displacement will be accumulated. The total permanent displacement is therefore not a purely function of the nature of the soil, but it also depends on the characteristics of the ground shaking. A detailed seismic assessment of the bridge completed in 2004 predicted slope failures at ground accelerations greater than 0.12g (Palermo *et al.* 2011). The shallow slope failures observed following the Christchurch earthquake were therefore expected although the slopes performed better than predicted. In order to assess the real performance of the bridge, the slope failure is thus simulated adding to the transitory displacement at the abutment (ground motion), the permanent displacement (slope failure). Since in the transversal direction the ground is confined, only the permanent displacement in the longitudinal direction of the bridge is considered.

Figure 6a shows the accelerograms for soil (continuous line) and structure (dashed line) for the two components of CMHS record, in the longitudinal axis of the bridge. The corresponding relative structure displacements are shown in Figure 6b. The displacements in the same direction are added together to find the total displacement of each abutment equal to 20.11 cm.

As already explained in the previous paragraph, the movement of the abutments caused shearing of two of the bolts on the new linkage brackets. To estimate the he maximum load capacity taken by the vertical bolts, the connection between the deck and the abutments has been represented by two multi-spring elements: one for the existing holding down dowel connecting the diaphragm to the abutment, and the other one for the strengthening rods put into the bottom of the beams. The horizontal linkages activate only when subjected to tensile forces and so their function is to prevent any possible unseating of the deck to the abutments.

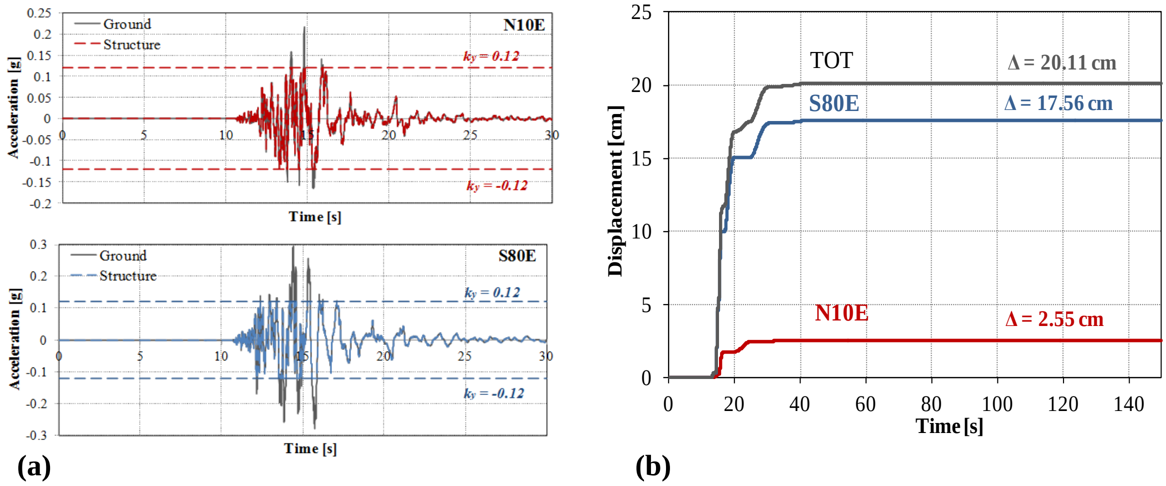

Figure 6. a) Acceleration Time Histories for ground and structure, CMHS components; b) Displacement Time History relative to soil, CMHS records

Since the vertical linkages were designed to sustain shear forces, the translational stiffness of the rod, liken to a double bending beam, was considered.

The drawings show that the extisting holding down dowels have a total vertical length of 4' (~1.2 m) with a stretch 1' long (~ 300 mm), between the abutment and the diaphragm, in a hole filled with a plastic material "Pliastic". Considering that in this stretch the rod is free to deform, the length of the beam for the existing linkages was assumed to be 300 mm.

The determination of the actual length of the beam for the retrofit bolts was less immediate. As shown in Figure 7a, they are fully fixed in the concrete all the length of the bolt. For numerical analyses a length of 30 mm was assumed.

Results of the analysis show that, adopting an elasto-plastic rule to represent the behaviour of the bolts, it was possible to pick out the brittle failure of retrofit bolts, as expected from a shear failure, at ~1120 kN (sum of four bolts). Figure 7b shows the shear force time history with the indication of the instant when the shear failure occurs (~11 s).

Although retrofit work did not perform well, if the deck did not tie together with the abutments, the inertial forces of the deck and abutment-soil would cause severe pounding between the deck and the abutment damaging the superstructure.

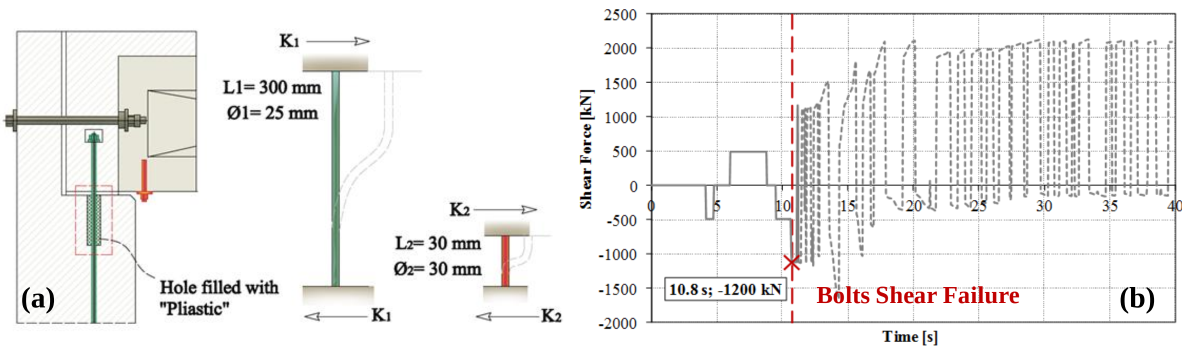

Figure 7. a) Schematization of the bolts for numerical analyses; b) Time History of the Shear Force acting on retrofit bolts, with the indication of the breaking point.

4.3. Moorhouse Ave Overpass

4.3.1. Description of the structure and earthquake damage

Moorhouse Avenue Overpass (-43.5399, 172.6367) is 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 (Figure 8a). 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 (Figure 8b).

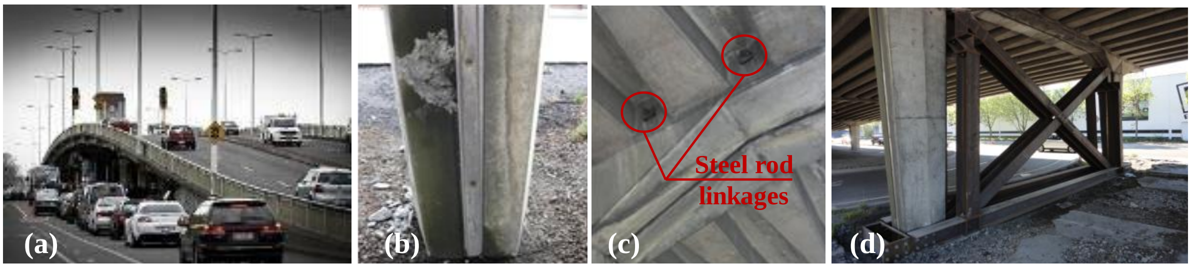

Figure 8. a) Overall view of the bridge; b) Pier expansion joints [Courtesy of OPUS]; c) Steel rod linkages [Courtesy of OPUS]; d) Temporary repair solution.

The performance during the February event was unsatisfactory. The mechanism of damage is shown in Figure 9a: west piers were significantly damaged with shear cracking and buckling of longitudinal rebars. The bridge sustained damage to one column near the North-East approach where a deck expansion joint was located. 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 (Figure 8c): 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 9b).

Observations after the Christchurch event indicated that the damaged columns had started to buckle putting the central span at risk of collapse (Figure 9c). 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 9d).

To prevent further damage a repair solution consisting of dual cross-bracing units at each of the weakest piers was constructed (Figure 8d). The bracing was designed in order to have the same lateral stiffness of the existing piers and not modify the loading path in the other existing parts.

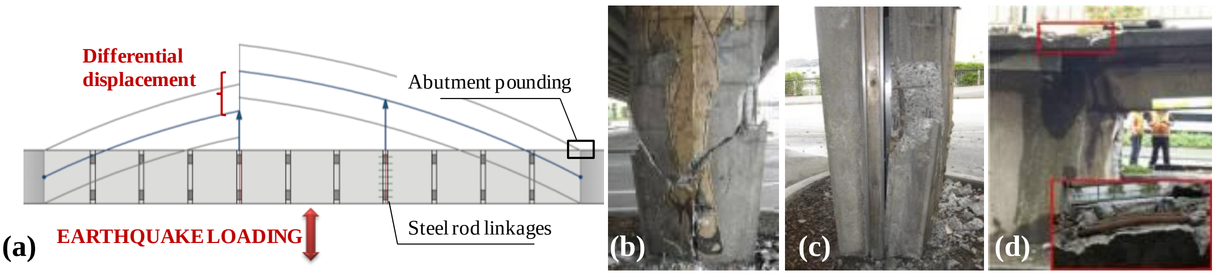

Figure 9. a) Plan on bridge showing qualitative displacement profile under transverse loading; b) Shear failure mechanism of the pier [Photo by A. Palermo]; c) Longitudinal bars buckling of the pier d) Concrete spalling and bar buckling at South-West side abutment [Photo by A. Kivell]

4.3.2. Performance assessment

Numerical analyses with Ruaumoko 3D (Carr 2008) were undertaken to assess the performance of the bridge, by considering two models: an As-Built model based on the design drawings of the bridge and a Repaired model based on the bridge after temporary repair. The records from the CCCC station were used for the analyses. The As-Built model was subjected to the ground motions recorded in both the Darfield and Christchurch earthquakes. The repaired model was subjected only to the ground motion recorded during the Christchurch earthquake for comparison against the As-Built model.

Figure 10a shows the peak displacement profiles of each model: the pier located at the expansion joint without the steel rod linkages underwent the largest displacement demands in the As-Built model during the last seismic event, reaching 9.85 mm; in fact it exhibited the most severe damage in assessment inspection. A shear failure occurred in the As-Built model after 11.2 s when subjected to the Christchurch earthquake as expected (Figure 10b). No failure occurred in any of the other analyses, indicating that the repair method was effective in preventing failure of the bridge. The displacement profiles show a larger peak displacement in the South direction than the North direction which is consistent with the damage to the west abutment where spalling occurred on the Southern side of the abutment, but not the Northern side. Moreover the peak displacement and shear force of the As-Built model during the Christchurch earthquake were approximately 1.6 times those that occurred during the Darfield earthquake.

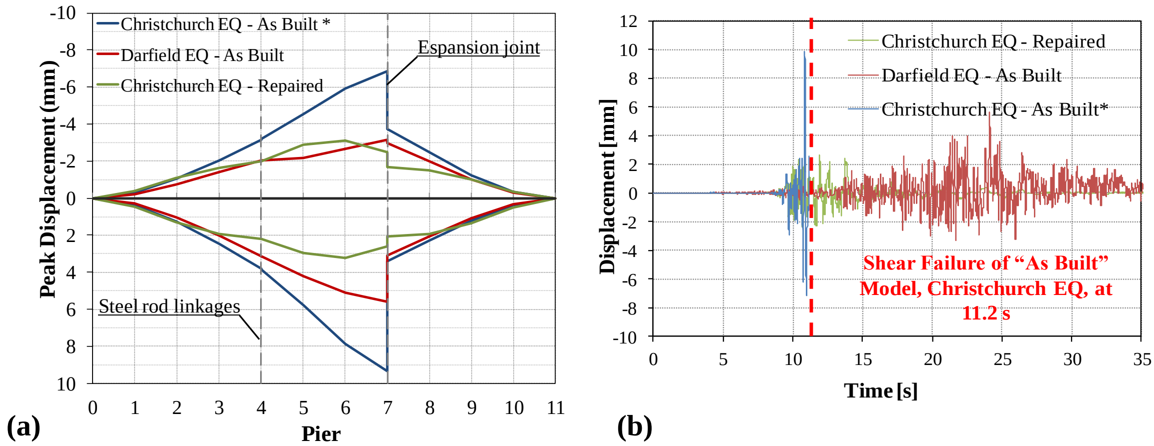

Figure 10. a) Displacement profile of each model; b) Time History displacement with the indication of the instant in which the shear failure happened according to the numerical analyses.

5. CONCLUSIONS

The performance of bridges during the 22nd February 2011 Earthquake was satisfactory. This was confirmed by the results of the database of the University of Canterbury which was compiled in order to offer an unbiased statistical tool for assessing the bridges of the city. In fact although the Christchurch earthquake was more than 500 years return period design event according to (NZS 1170.5, 2004), only 4% of the entire stock was considered severely damaged.

The numerical analyses also confirmed what was observed during the inspections after the earthquake. At Port Hills Overpass the linkage retrofitting probably prevented significant damage to the span to pier connections and to the base of the piers. Nevertheless the analyses demonstrated that the deck's flexibility caused a high displacement demand at the central pier, resulting in bar buckling and concrete spalling. Moreover the results showed that the high vertical acceleration had an important influence on the response of the structure as result of the considerable variation of the moment-curvature capacity of the piers.

With regard to Moorhouse Overpass, the collapse of the western pier was found to be caused by shear failure and secondary buckling interaction. The "repaired" condition with addition of cross-bracing, which re-established the symmetry in both 'split' piers, showed a large decrease of member loads throughout the structure,. This reduced transversal displacements by 70% proving that the repair is effective in preventing future damage.

Lastly at Horotane Valley Overpass, being the bridge very stiff, the slope failure of the approach was the main cause of the shear rupture of the retrofit bolts, Despite the failure, the retrofitting was clearly beneficial and probably prevented the onset of damage to the beam diaphragms due to possible pounding with abutments.

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