Damage assessment, analysis and modelling of bridges in non-liquefiable soil during Canterbury earthquakes

2012 NZSEE Conference

A. Palermo & M. Brando

Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand

E. Camnasio

Department of Structural Engineering, Politecnico di Milano, Milan, Italy

ABSTRACT: On February 22, 2011 a moment magnitude M_w 6.2 earthquake occurred with an epicentre near the town of Lyttelton, 10 km South of the Christchurch Central Business District (CBD). Though the majority of the observed damage was due to liquefaction and lateral spreading of the river banks, examples of significant damage occurred to bridges on non-liquefiable sites as well. A brief summary of field observations is presented herein, highlighting the main damage typologies shown by the bridges with respect to each structural component. The paper focuses on the seismic performance of two concrete bridges: Moorhouse Avenue Overpass and Port Hills Overbridges. The assessment involved site investigations and numerical modelling, including both quasi-static and dynamic analyses. The models include features such as shear and axial bending interaction which were essential factors to properly capture the seismic performance of the bridges during Canterbury earthquakes. The results of the analyses are consistent with the observed damage.

1 INTRODUCTION

The surroundings around Christchurch contain more than 800 road, rail and pedestrian bridges. Over half of the bridges are integral reinforced concrete or hybrid, i.e. precast concrete beams and cast in situ substructure, with the remainder consisting of timber and steel construction. The February 22, 2011 earthquake had a moment magnitude M_w 6.2, which resulted in strong ground shaking in the Central and Eastern regions of Christchurch, with the majority of severe bridge damage localized in this region. Most of the damage was a result of liquefaction and lateral spreading of the river banks, with very few examples of significant bridge damage on non-liquefiable sites (Wotherspoon *et al.* 2011).

Only a few bridges suffered significant visible structural damage as a result of ground shaking, even if the damage threshold level was much lower than the estimated bridge response accelerations in the earthquakes. Their typical monolithic construction and axial strength also meant that bridges designed to old codes (pre 60s, 70s) were able to resist the axial demands placed on the structure due to lateral spreading, even though they were not specifically designed for these seismic actions.

This paper presents a general overview of the damage suffered by bridges during the February 22, 2011 earthquake and a seismic analysis of the Moorhouse Avenue Overpass and Port Hills Overbridges. These bridges are key links in the arterial road routes of Christchurch city. Numerical modelling which includes shear and axial bending interaction (Priestley *et al.* 1996, Bresler 1960) is adopted herein. Nonlinear time-history analyses which replicate the effects of Christchurch earthquake are compared with the damage observed during the bridge inspections.

2 SEISMIC DEMAND

The M_w 6.2 February 22, 2011 Christchurch earthquake had an epicenter 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 compared to the Darfield event in September 2010 (Palermo *et al.* 2010). Further aftershocks occurred during the following months, with one of the strongest on June 13, 2011, with an epicenter again on the South Eastern edge of the city and a moment magnitude M_w 6.0 (GNS Science 2011).

Horizontal Peak Ground Accelerations (PGAs) were in the range of 0.37-0.51g in the Christchurch CBD. Strong motion records indicated that most of the bridges within 10 km from 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 epicenter 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 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 soil type D, 500 year return period.

Figure 1b indicates the locations of the road bridges most severely damaged in the Christchurch earthquake, which are located to the zone affected by moderate to severe liquefaction. Most of Central and Eastern Christchurch area was identified as having high liquefaction risk (Wotherspoon *et al.* 2011). The damaged bridges were mostly located along the Avon River, which area was affected by moderate-severe liquefaction. Although bridges crossing the Heathcote River were closer to the fault rupture, they suffered much less damage being the liquefaction phenomena less significant in this area.

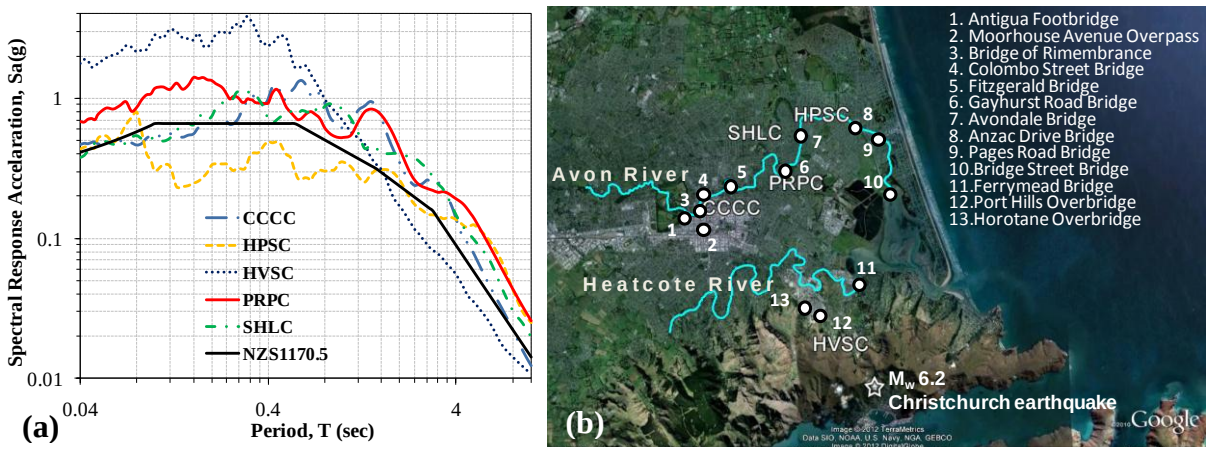

Figure 1. (a) Response spectra of the geometric mean of the horizontal accelerations at strong motion stations in Central and Eastern Christchurch compared to NZS1170.5 design response spectrum for Christchurch, site subsoil class D for a 500 year return period. Four letter symbols represent different strong motion stations. (b) General view of Christchurch and the surrounding region, with locations of a selection of damaged bridges, strong motion stations and the epicenter of the major event(Google Inc. 2011).

3 EARTHQUAKE DAMAGE

This section gives a brief summary of the most significant damage to bridges during Canterbury earthquakes (Palermo *et al.* 2011). Liquefaction and lateral spreading were the main causes of bridge damage. However, bridges suffered moderate damage, exhibiting generally a sturdy seismic response. Nevertheless, because some bridges were critical links to the city infrastructure network extensive traffic disruption was evident immediately following the events.

3.1 *Abutments and foundations*

Abutments presented two different types of damage, i.e. residual displacements/rotations of the substructure and structural damage (cracks, concrete crushing and spalling) to abutments and piles, both caused by lateral spreading of the approaches and river-banks. The abutments tended to rotate backwards toward the riverbanks or converge inwards, meaning that the bridge superstructure acted as a rigid strut while the foundations underwent forced rotations or movements in the direction of lateral

ground flow.

During the Darfield earthquake the Western abutment of Bridge Street Bridge (latitude: -43.5252, longitude 172.7241) rotated by approximately 5 degrees due to lateral spreading, and light cracking was observed on the tension face of the abutment piles after the event. The pile damage was exacerbated during the Christchurch earthquake, with further abutment rotation to more than 12 degrees at the West end (Figure 2a), and plastic hinging was clearly visible on the abutment piles of the Western abutment (Figure 2b).

3.2 *Piers*

Pier damage was caused either by ground shaking, with a combination of vertical and horizontal components, or by lateral spreading. Some bridges experienced pier cracking as a result of extensive lateral spreading, which caused a lateral force applied to the pier base inducing a large moment at the stiff pier-deck interface. The liquefied soil layer also reduced the lateral stiffness of the pier foundation system, allowing rotation of the bottom of the pier towards the centre of the river. Ferrymead Bridge (-43.5584, 172.7086) sustained damage due to lateral spreading, with permanent rotation and cracking of some of the piers situated in the estuary (Figure 2c-d).

Figure 2. Bridge Street Bridge. (a) Back rotated abutments [Photo by M. Bruneau]; (b) Plastic hinging of the abutment piles of Bridge Street Bridge [Photo by M. Bruneau]. Ferrymead Bridge. (c) Tilted piers [Courtesy of OPUS]; (d) Close-up view of cracks caused by superstructure bearing on the edge of the pier top [Courtesy of OPUS].

4 CASE STUDIES

This section presents a detailed seismic analysis of two highway bridges, i.e. Moorhouse Avenue Overpass and Port Hills Overbridges (Figure 3), which were mainly damaged by the ground shaking. The bridges are modeled by using Ruaumoko 3D (Carr 2008). The piers are modeled as Giberson elements with lumped plasticity (Giberson 1967). The Takeda hysteresis model (Otani 1974) is adopted to describe the cyclic behavior of the plastic hinges at the ends of the piers. Nonlinear dynamic time-history analyses are then carried out using the ground motions recorded during the Canterbury earthquakes.

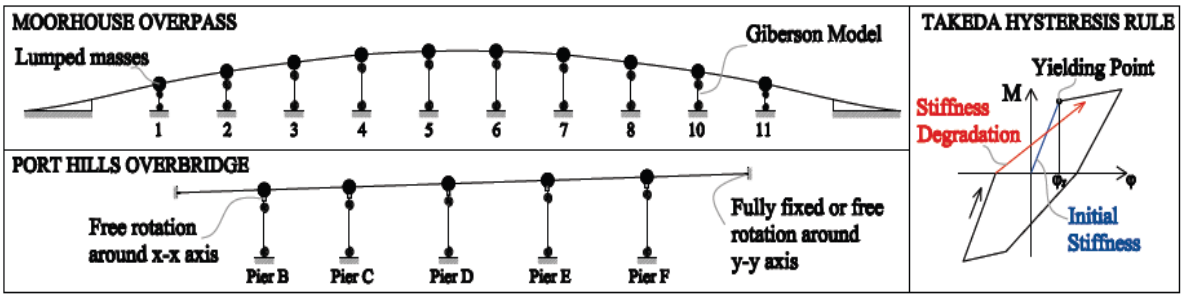

Figure 3. Sketches of the numerical model of the bridges with indication of the hysteresis model adopted for the piers.

4.1 *Moorhouse Avenue Overpass*

4.1.1 *Description of the structure*

Moorhouse Avenue Overpass (-43.5399, 172.6367) is an eleven span reinforced concrete structure providing grade separation between Moorhouse Avenue and Colombo Street. Moorhouse Avenue

itself is one of the four avenues that encase the CBD, 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 sections, separated by expansion joints (Figure 4).

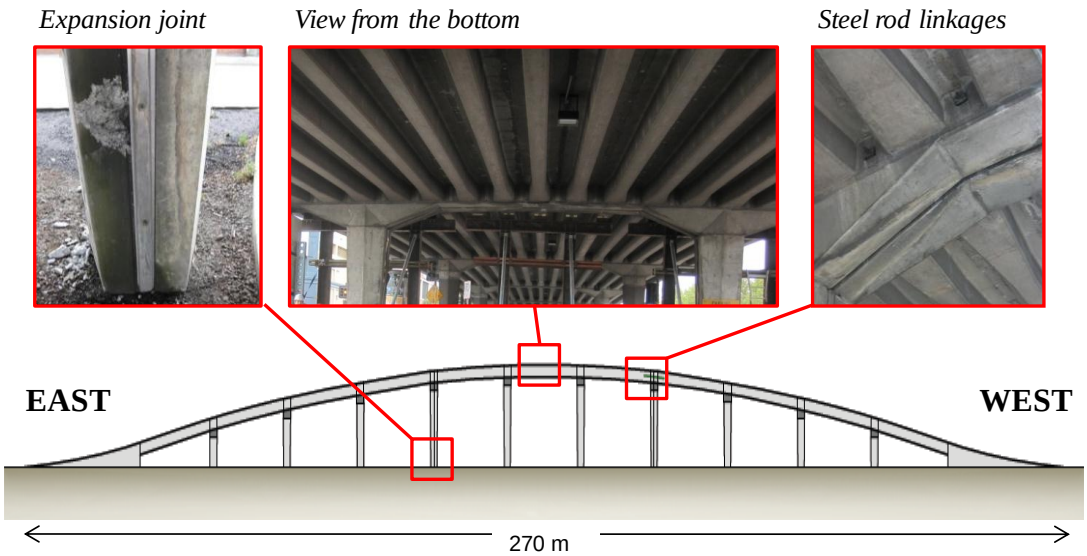

Figure 4. Moorhouse Avenue Overpass. Sketch of the bridge elevation with location of the expansion joints and steel rod linkages (Palermo* et al. *2011).

4.1.2 *Earthquake Damage*

During the February earthquake, 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 at the expansion joint at the only West side of the bridge caused irregularity in the bridge structure (Figure 4). In fact, because of the West and central part of the linked bridge, 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 5a).

Figure 5. (a) Shear failure mechanism of the pier [Photo by A. Palermo]; (b) Concrete spalling and bar buckling at South-West side abutment [Photo by A. Kivell]; (c) Temporary repair solution [Photo by A. Palermo].

Observations after the Christchurch event indicated that the damaged columns had started to buckle putting the central span at risk of collapse. 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 5b). Strengthening works were erected around the failed pier. These consisted of two built up square block concrete columns with timber blocking to prop the bridge while further works were undertaken. A multi-span structural steel frame was designed and constructed spanning between the two failed columns, as a temporary solution to prevent the collapse of the damaged section. This solution did not provide any additional lateral stability and therefore the overbridge was not reopened to vehicle traffic until March 31, 2011, when the final repair solution consisting of dual cross-bracing units at each of the weakest piers was constructed (Figure 5c).

4.1.3 *Performance Assessment*

In order to assess the performance of the bridge, a numerical analysis of Moorhouse Avenue Overpass is carried out on 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 As-Built model is subjected to the ground motions recorded in both the Darfield and Christchurch earthquakes. The repaired model is subjected to the ground motion recorded during the Christchurch earthquake only.

Figure 6a shows the peak displacement profiles for each model. The pier located at the expansion joint without the steel rod linkages undergoes the largest displacement demand in the As-Built model; it actually exhibited the most severe damage in assessment inspection. As expected, a shear failure occurs in the As-Built model when subjected to the Christchurch earthquake (Figure 6b). No failure occurs for the Repaired model, indicating that the repair and retrofit methods were effective in preventing the failure of the bridge. The insertion of tie-bolt links in the free expansion joint would also have limited the displacement demand in the piers and re-established the regularity of the bridge response under transverse seismic loads. The displacement profiles show a larger peak displacement in the South direction than the North direction which is consistent with the observed damage to the West abutment, where spalling occurred on the Southern side of the abutment, but not on the Northern side. Moreover, the peak displacement and shear force of the As-Built model during the Christchurch earthquake is approximately 1.6 times those that occurred during the Darfield earthquake.

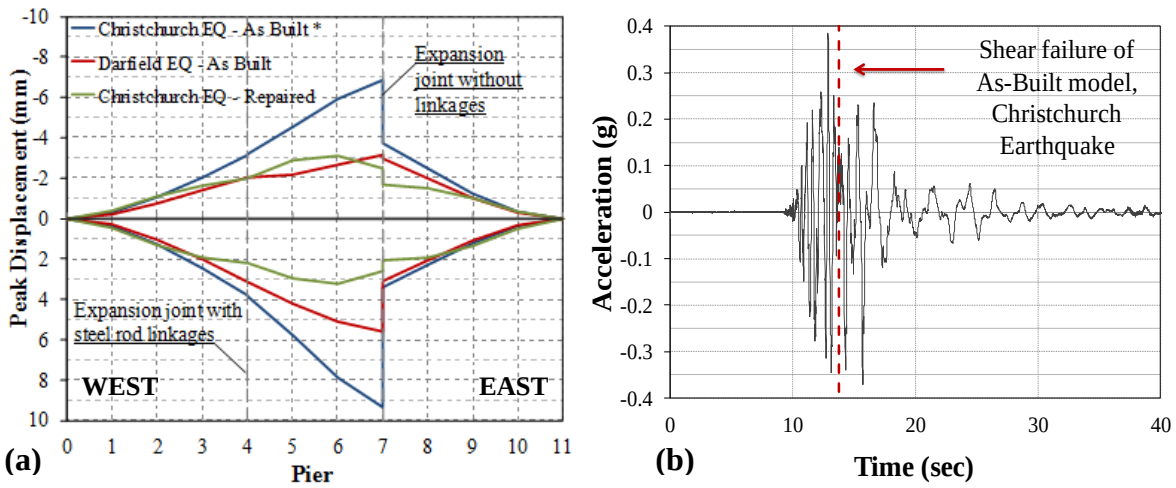

Figure 6. (a) Displacement profile of each model; (b) Ground motion measured at Catholic Cathedral College (CCCC) with the indication of the instant in which the shear failure occurred according to the numerical analyses.

4.2 *Port Hills Overbridges*

4.2.1 *Description of the structure*

The Port Hills Overbridge (-43.5711, 172.6934), constructed in 1963, are six simply supported span bridges (Figure 7). The prestressed concrete log type beams are connected by a cast in situ concrete slab. The spans vary in length between 9.4 and 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 and founded on spread footings. As part of seismic retrofit programme (Palermo *et al.* 2010), the bridges were recently strengthened by fixing fabricated steel shear keys to the underside of the beams at both the abutments and the 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. Linkage at the abutments was provided by rods extending between the brackets and the soil face of the abutment seating beams. 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 which shortened its flexural height. By sleeving the piers, the unbalanced distribution of stiffness in the structural system was removed

Figure 7. Overall view of Port Hills Road Overpass [Courtesy of OPUS].

4.2.2 *Earthquake Damage*

During the Christchurch earthquake, flexural cracking developed in the lower halves of the piers of both bridges except those adjacent to the abutments. In fact, despite the presence of the linkage rods increased the deck diaphragm action, this was not enough to control the higher displacement demand 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 8a). The damage increased during the June 13, 2011 aftershock. The nominal 10 mm 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 abutment. There was also minor settlement and displacement of the approach pavement and back-fill at the abutments.

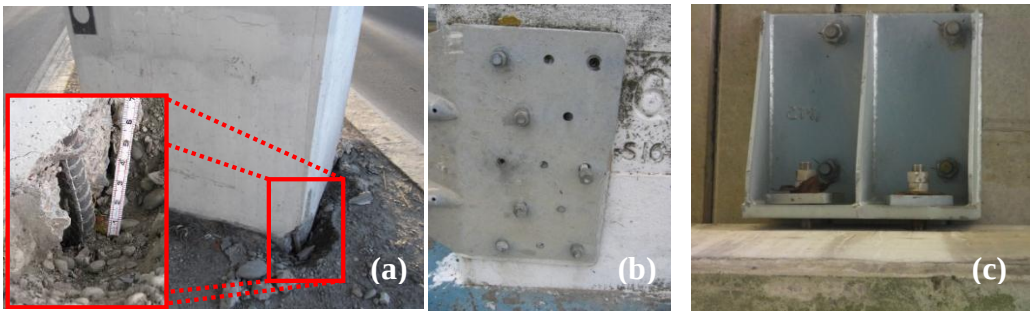

Figure 8. Port Hills Overbridge. (a) Buckling of reinforcing steel at the base of the piers [Courtesy of OPUS]; (b) Pop out of the bolts of the guardrail [Courtesy of OPUS]; (c) Elongation of the links between the spans [Courtesy of OPUS].

4.2.3 *Performance Assessment*

In order to assess the performance of the bridge during the February 22, 2011 earthquake, time-history analyses are carried out applying to the structure the accelerograms recorded by the nearest ground motion stations, i.e. Heathcote Valley Primary School (HVSC) station, Christchurch Cashmere High School (CMHS) and Catholic Cathedral College (CCCC). The piers are considered fully fixed at the base while at the top they can rotate about the transverse axis. This assumption seems to be consistent with the damage observed in the piers. Because of the presence of linkage rods, the deck is assumed continuous. Two different deck-abutment connections are considered, i.e. a fully fixed connection or a pinned connection with respect to the vertical axis. The second assumption is adopted in order to model the observed relative movement between the deck and the approaches which resulted in some cracks transverse to the bridge axis.

While the CCCC and CMHS records match the design spectrum (NZS 1170.5:2004) quite well in the range 0.17-0.27 s (natural period of the structure), the spectral accelerations from HVSC are much greater than the design ones, reaching a value of 1.2g (Figure 1a). In fact, this event caused exceptional vertical accelerations with values up to 2.2g at the HVSC station (Palermo *et al.* 2011). This led to a significant variation of the axial force in the columns (Figure 9a), which caused a reduction in the ductility of the piers when subjected to high axial compression as shown in Figure 9b for the central pier.

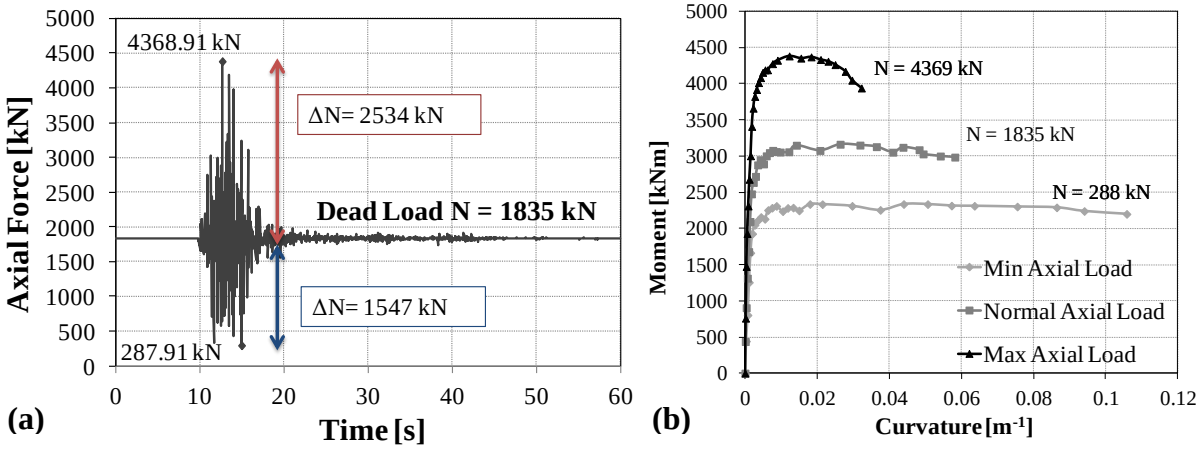

Figure 9. (a) Time-history of the axial force loading the central pier when subjected to HVSC record. (b) Evolution of the moment-curvature with varying axial force.

Time history analyses are carried out including the vertical component of the ground motion. Figure 9b shows how the moment capacity and curvature ductility in the central pier changes drastically. When the axial load variation caused by the vertical ground acceleration (HVSC record) is additive to the static load (N+ΔN), there is an increment of 39% in the moment capacity a reduction of 49% in the curvature ductility, relative to the case in which the acceleration-induced axial load variation is not considered. Vice versa if the axial load variation is subtracted from the static load, (N-ΔN) the moment capacity reduces of 28% while the curvature ductility increases of 94%.

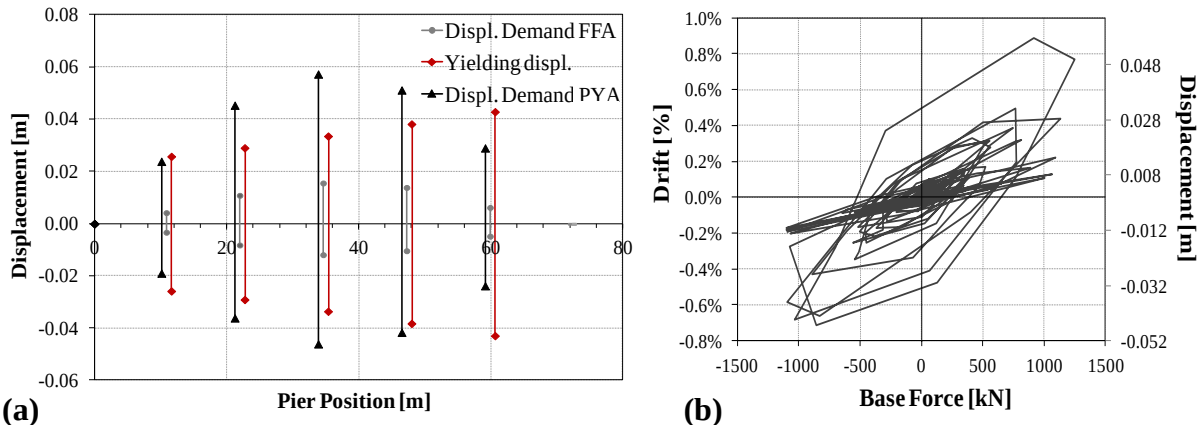

Figure 10. (a) Comparison between the displacements of the top of the columns subjected to HVSC records; (b) Base shear vs top displacement of the central pier when subjected to HVSC records in the PYA model.

Numerical results show that the bridge performed well when subjected to the CCCC and CMHS records (with vertical acceleration of 0.8g), during which they remained in the elastic shape. Nevertheless, the observed actual damage showed that the central piers behaved unelastically, even if the overall performance was better than expected from the high spectral acceleration values. This inconsistency can be due to the location of the strong motion stations, which were far from the bridge and the epicentre. The accelerograms recorded by these stations were chosen to perform the dynamic analyses because the soil conditions at the station site are similar to the ones at the bridge site. However, the PGA recorded at these stations could have been smaller than the one experienced by the bridge.

A good correspondence between the numerical analyses and the actual response of the bridge is obtained with the HVSC record. Figure 10a shows that the displacement demand is lower than the displacement at the yield of the base section of the piers for the model with Fully Fixed Abutments (FFA). On the other hand, in the model with the superstructure Pinned around the Y-y Axis (PYA) plastic hinges (Figure 10b) develop at the bottom of the piers, with top displacements greater than the yielding ones (Figure 10a). The response of the bridge is hence likely to have been more similar to the

numerical response given by the second model, being the bridge prefabricated with post-installed mechanical connections. In fact, the fully fixed model overestimates the stiffness of the retrofit linkages.

5 CONCLUSION

The Canterbury bridges performed generally well during the 2010-2011 earthquake sequence. The robustness of Christchurch City Council road bridges built in the 1940s to 60s without any seismic design criteria helped to sustain earthquake loadings comparable or higher than the current design levels. The numerical models were able to capture the observed behavior of the two bridges herein investigated. At Moorhouse Overpass, the collapse of the pier at the expansion joint was caused by shear failure and secondary buckling interaction. As far as the Port Hills Overbridges, the shear keys and the rod linkages made the deck stiffer. Despite the retrofit intervention, the analyses proved that a higher displacement demand occurred in the central piers, due to the flexibility of the deck. More importantly, the high vertical accelerations varied the moment-curvature capacity of the piers drastically, thus causing increased displacement demand too. Despite the simplified assumptions of the used numerical models and the complexity of the two case studies proposed herein, the analyses were quite consistent with the actual performance of the structures. The results of these analyses could be useful for the Christchurch City Council and the New Zealand Transportation Agency as complementary assessments for the further retrofit actions on these bridges.

ACKNOWLEDGEMENTS:

The authors wish to acknowledge the following institutions and consulting engineering companies for their collaboration, technical assistance and data supply: NZTA (John Reynolds), Christchurch City Council (Lloyd Greenfield, Stewart Smith, David McNaughton), Opus International Consultants Ltd (Mike Cowan and Anthony Rooke). We also acknowledge Samuel White for his help on analyses of Moorhouse Overbridge and Prof. Athol Carr for his precious suggestions during the modelling phase.

REFERENCES:

Bresler, B. 1960. Design Criteria for Reinforced Columns Under Axial Load and Biaxial Bending, ACI Journal, 32(5), pp. 481-490.

Carr, A.J., 2008. *RUAUMOKO Program for Inelastic Dynamic Analysis*. Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

GeoNet 2011. Strong motion FTP site, available from ftp://ftp.geonet.org.nz/strong/processed/Proc/2011/02_Christchurch_mainshock_extended_pass_band/(Accessed August 12, 2011).

Giberson, M., 1967. The response of nonlinear multi-storey structures subjected to earthquake excitations. *Earthquake Engineering Research Laboratory*, Pasedena.

GNS Science 2011. http://www.geonet.org.nz/earthquake/historic-earthquakes/ GeoNet – Historic Earthquakes. (Accessed September 1, 2011).

NZS 1170.5:2004. *Structural Design Actions. Part 5: Earthquake actions – New Zealand*.

Otani, S. 1974. Inelastic analysis of R/C frame structures. *ASCE Journal of the Structural Division*, 100(7)

Palermo, A., Le Heux, M., Bruneau, M., Anagnostopoulou, M., Wotherspoon, L., Hogan, L. 2010. Preliminary findings on performance of bridges in the 2010 Darfield earthquake. *Bulletin of the New Zealand Society for Earthquake Engineering*, 43(4), pp. 412-420.

Palermo, A., Wotherspoon, L., Hogan, L., Le Heux, M., Camnasio, E. 2011b. Seismic Performance of Concrete Bridges during Canterbury Earthquakes. *Structural Concrete*, 12(4).

Priestley M. J. N., Seible F., Calvi G. M., 1996. *Seismic Design and Retrofit of Bridges*, John Wiley & Sons, New York.

Wotherspoon, L., Bradshaw, A., Green, R., Wood, C., Palermo, A, Cubrinovsky, M., Bradley, B. 2011. Performance of Bridges during the 2012 Darfield and 2011 Christchurch Earthquakes. *Seismological Research Letters*, 82(6), pp. 950-970.