Seismic Assessment of New Zealand Bridges in non liquefiable soils during Canterbury earthquake sequence

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Summary

On February 22nd, 2011 a Mw6,2 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, has been documented and collected into a Bridge Database 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. Detailed inspections supported by numerical analyses were carried out. The results are consistent with the damage observations and highlight unexpected design issues, such effects of vertical accelerations, slope failure stability, shear-flexure interaction.

Keywords: Canterbury earthquakes, bridges, damage assessment, time history analyses.

1. Seismic Demand and Post-Earthquake Bridge Damage

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]. Horizontal PGAs were in the range of 0,37-0,51g in the Christchurch CBD, while vertical PGAs reached up to 2,1g.

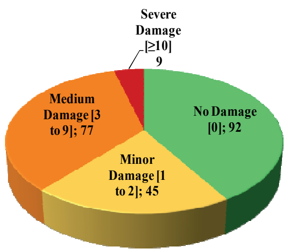

Figure 1: Damage severity to the Canterbury Bridge stock following 22nd February 2011 Christchurch earthquake.

This shaking level combined with the soil characteristics of the region caused extensive liquefaction and lateral spreading, especially close to the river-banks [2].

Following the earthquake, all bridges of the city were inspected by the practitioners and by researchers. The information was then collected in a database, coordinated by the University of Canterbury, which offers an unbiased method for assessing the overall performance of bridges in Canterbury. Results are shown in Figure 1. The general bridge performance during the earthquake was satisfactory, with only 4% of bridges sustaining severe damage [3]. The results also confirmed the lateral spreading as one of the main causes of the structural and non-structural damage. It appeared to be more sever for post 60s precast bridges. On the other hand, the robustness of some integral monolithic Christchurch City Council road

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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). This severely impacted on the bridge network compromising its regular functionality for several months. Improvements in this sense need to be done as New Zealand Standards as well overseas codes are deficient in terms of design integrated approach.

2. Performance of Highway and Road Bridges in non liquefiable soils

As support to Christchurch City Council and New Zealand Transportation Agency (NZTA) for their repair/retrofit strategy, three critical bridges, not affected by liquefaction, were numerically seismic assessed: Port Hills and Horotane Valley Overbridges and Moorhouse Ave Overpass.

The analyses of Port Hills Overpass demonstrated that the deck's flexibility caused a high displacement demand at the central pier, resulting in bar buckling and concrete spalling (Figure 2a). 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. 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 (Figure 2b). With regard to Moorhouse Overpass, the collapse of the western pier was found to be caused by shear failure and secondary buckling interaction (Figure 2c). The results from the analyses were then consistent with the damage observations and highlighted unexpected design issues, such effects of vertical accelerations, slope failure stability, shear-flexure interaction. The unique findings from this reconnaissance experience and the numerical analyses may become relevant for also further implementation or improvement of European Standards, in particular Eurocode 8, part. 2.

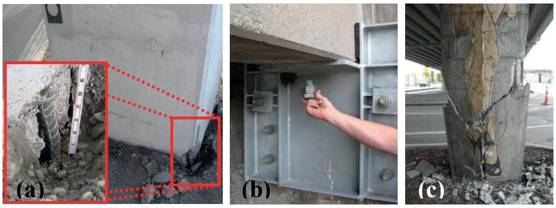

Figure 2: a) Buckling of reinforcing steel at the base of the central pier of Port Hills Overpass; b) Sheared bolt at the abutment retrofit of Horotane Valley Overpass; c) Shear failure of the column of Moorhouse Ave Overbridge

3. References

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