STRUCTURAL AND GEOTECHNICAL ASPECTS OF THE CHRISTCHURCH (2011) AND DARFIELD (2010) EARTHQUAKES IN N.ZEALAND

Eleni Smyrou[1], Panagiota Tasiopoulou[2], İhsan E. Bal[3],
George Gazetas[4] and Elizabeth Vintzileou[5]

ABSTRACT

Yeni Zelanda'nın Christchurch şehri biri Eylül 2010 ve diğeri Şubat 2011'de olmak üzere iki ciddi deprem ile sarsılmıştır. Bu depremlerin ikisi de mühendislik camiası tarafından daha önceden bilinmeyen faylar üzerinde meydana gelmiştir. İlk deprem ciddi hasarlara neden olmasına rağmen can kayıpları yaşanmamış, ikinci depremde ise 172 insan hayatını kaybetmiştir. Gözlenen hasarlar bölgedeki yapı stoğunun özelliklerine bağlanabileceği gibi çok geniş bir alana yayılan sıvılaşma hasarları ile de kısmen açıklanabilir. Bu makalede bu iki depremde yapılar ve zeminin davranışı ve bu ikisinin etkileşimi konu edilmiştir. Bu çalışmada, gözlenen hasarı mevcut deprem ivme kayıtları ve zemin hasarları ile ilişkilendirebilmek için uğraşılmıştır. Buradan elde edilen sonuçlara göre, özellikle şehrin merkezinde orta katlı ve yüksek yapılarda gözlenen hasar ve yıkımların, depremin spektral büyütmelerinin uzun periyot bölgesinde şehir merkezinde konsantre olması ile açıklanabileceği sonucuna varılmıştır. Ayrıca sıvılaşma hasarlarından en çok etkilenen köprü titpi yapıların her iki depremdeki hasar durumları da ayrıntılı olarak sunulmuştur.
Anahtar Kelimeler: Christchurch depremleri, Sıvılaşma, Deprem hasarları, Yüksek yapılar, Köprüler

ABSTRACT

The city of Christchurch, New Zealand, was hit by two severe earthquakes in September 2010 and February 2011. Both earthquakes were generated by faults which were completely unknown. The first earthquake, centred in sparsely-populated countryside, inflicted serious damage but no life losses, whilst the second earthquake in close proximity to the city caused 181 fatalities. The damage and some collapses observed during Darfield and Christchurch earthquakes can be attributed to the structural characteristics of the existing building stock in the region, as well as to the widespread manifestation of liquefaction. This paper aims to investigate aspects of these two earthquakes and their effects on soil and structure. An effort is made to relate the extensive structural damage to the observed soil behaviour and the particular features of the recorded motions. Certain structural types are examined with respect to the acceleration and displacement demands imposed by the two events, in an attempt to explain their behaviour and thus their performance. Bridge structures, which suffered significantly as result of soil liquefaction and lateral spreading, are also studied comparing their behaviour in the two consecutive earthquakes.
Keywords: Christchurch earthquakes, Liquefaction, Earthquake damages, Tall structures, Bridges

[1] PhD, Researcher, National Technical University of Athens, Greece, smiroulena@gmail.com
[2] PhD Candidate, National Technical University of Athens, Greece, ptasiopoulou@gmail.com
[3] PhD, Researcher, National Technical University of Athens, Greece, ihsanenginbal@gmail.com
[4] Professor, National Technical University of Athens, Greece, gazetas@ath.forthnet.gr
[5] Professor, National Technical University of Athens, Greece, elvintz@central.ntua.gr

INTRODUCTION

The city of Christchurch was hit on the 22nd of February 2011 by an earthquake of M6.3, the fault of which was in a distance of approximately 5-7km south of the Christchurch Central Business District (CBD). Due to its magnitude, shallow depth and proximity, as well as the topology characteristics, the earthquake caused a death toll of 181 people (as per 3rd of May 2011 according to the official list) and was proven particularly destructive for the CBD, the buildings in which suffered extensive damage. Since 22nd of February CBD remains excluded, all activities (commercial, financial etc) have ceased, while assessment and demolition procedures are still in progress. The Christchurch earthquake was preceded by a M7.1 earthquake event that occurred on the 4th of September 2010 in Darfield, Canterbury, on a fault extending 20-50km west of the city of. There were no casualties, but phenomena of widespread soil liquefaction and associated ground deformations and surface fault rupture augmented the damage reported in the wider area of Christchurch city and its environs.

The Canterbury Plains are covered with river gravels hiding the evidence for past-active, now buried, faults in this region. The newly-revealed Greendale fault was pre-existing, but unknown, and a patch was reactivated during the Darfield earthquake. There is no obvious fault structure directly connecting the faults that ruptured in the September'10 earthquake with the buried oblique thrust fault mechanism that generated February'11 event. The region remains active as shown by the intense sequence of aftershocks seismic events in Figure 1. For further information on seismological facts, the reader is referred to the work by Gledhill et al. (2011).

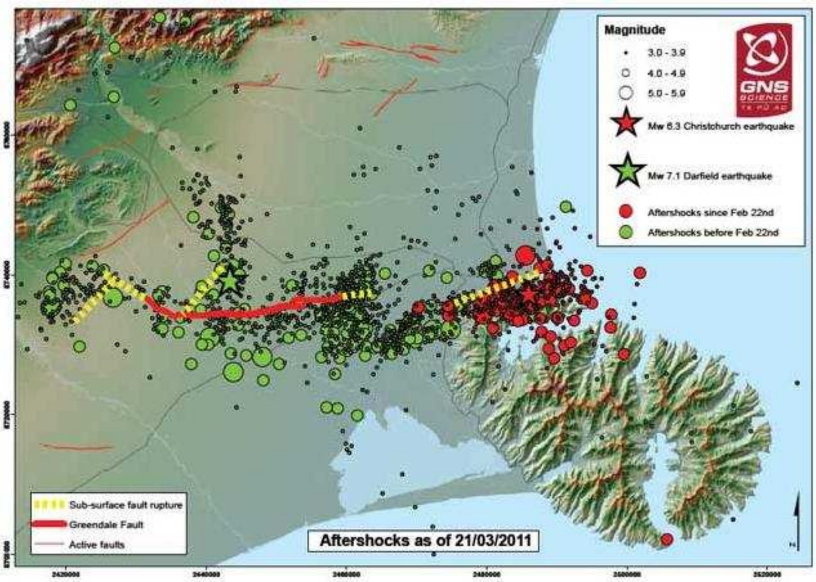


Figure 1. Map of the Canterbury region showing epicentres of the Darfield and Christchurch earthquake sequences. Map axes show N. Zealand Map Grid coordinates in meters. Subsurface rupture segments are based on geodetic and strong motion modelling, with the top edges of the ruptures mapped (Graphic by Rob Langridge and William Ries, GNS Science).

Thanks to a dense network of strong ground motion stations, a large number of records have been obtained, offering valuable information on the two events. Figure 2 presents the spatial distribution of the peak ground acceleration (PGA) values as characteristic indicator of the intensity of the Darfield and Christchurch earthquakes.

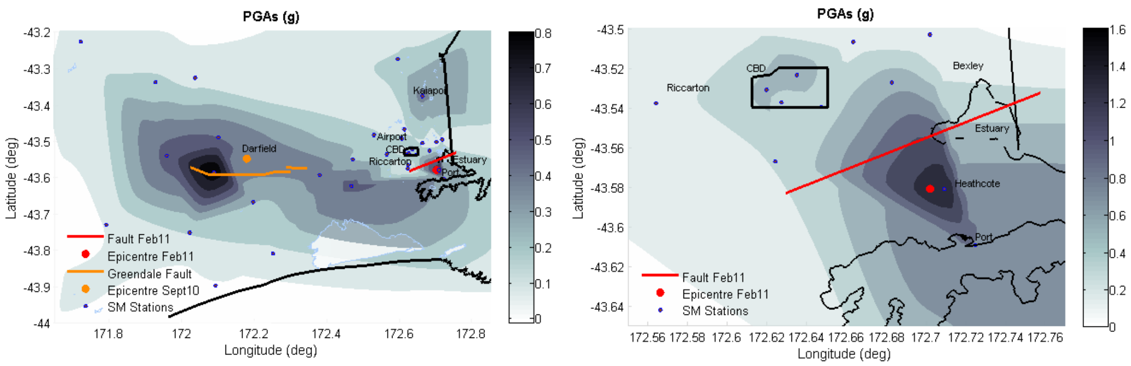


Figure 2. Approximate spatial distribution of the peak ground accelerations from the Darfield (left) and Christchurch (right) earthquakes.

EFFECTS OF THE CHRISTCHURCH EARTHQUAKES ON SOIL

The predominant geotechnical characteristic of both earthquakes events was liquefaction and lateral spreading. The Darfield earthquake caused significant liquefaction with evident signs of surface manifestation mostly in the eastern suburbs of Christchurch along the Avon river such as Avonside, Dallington, New Brighton and Bexley (Cubrinovsky et al., 2010). The CBD was much less affected by liquefaction. The subsequent Christchurch earthquake did not only re-liquefy the previously mentioned areas, but caused a more widespread liquefaction which also affected the southern suburbs and the CBD (Figure 3b). In particular, liquefaction in CBD was demonstrated by numerous sandboils formed in the perimeter of buildings and the large amount of sand emerging on to the surface.

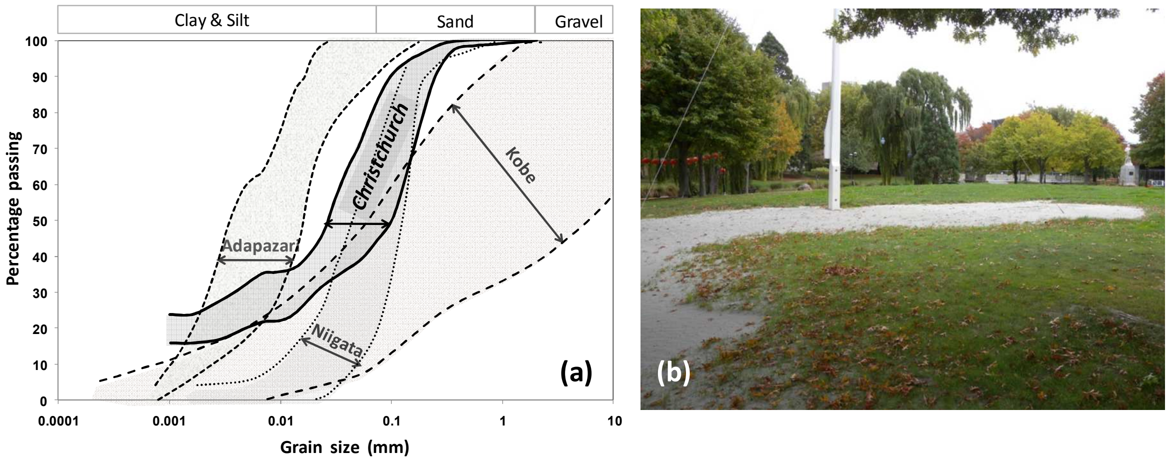
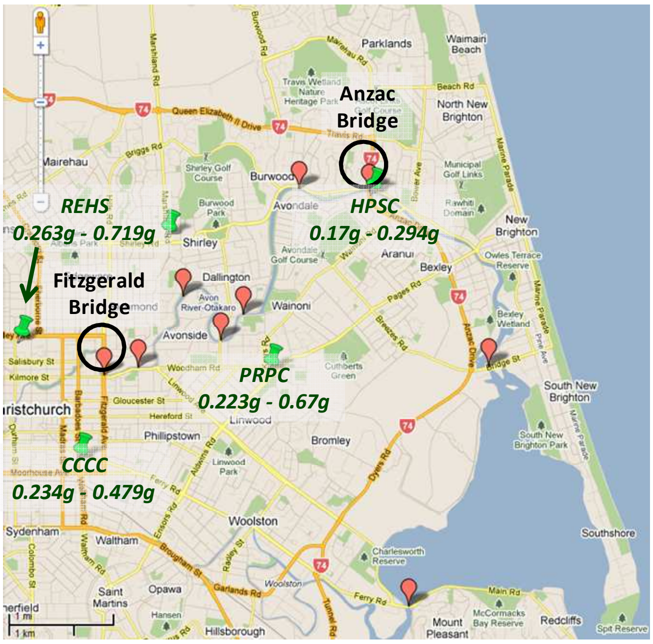


Figure 3. (a) Range of grain size distribution curves of soil samples from Dallington and Bexley areas after Darfield earthquake (Cubrinovski et al., 2010), compared with curves from Adapazari, Niigata and Kobe areas which have sustained liquefaction in past earthquakes. (b) Indicative photo of liquefaction along Avon River in CBD showing sandboils on the ground surface.

After Darfield earthquake several borehole tests were conducted covering a broad area of the eastern suburbs of Christchurch in order to investigate in detail the properties and the layering of the soil (Tonkin & Taylor LTD, 2011). The soil practically consists of layers of silty sand and clean fine to medium sand, with the ground water level reaching almost the ground surface (0.3 - 2.5m deep). The SPT and CPT values indicated very low cyclic shear resistance especially for the first 10m of soil. This suggests that the upper layers were those that mostly liquefied and explains why so large amount of sand reached the ground surface. Moreover, grain size distributions curves were produced for soil samples taken by Dallington and Bexley areas by sieve and hydrometer analyses after Darfield earthquake (Cubrinovski et al., 2010). Their range is depicted in Figure 3a and is compared with equivalent ranges from Adapazari, Niigata and Kobe areas which area believed to have sustained liquefaction in past earthquakes. All these facts explain the extensive liquefaction occurrence during the two earthquakes and especially the second one with significantly higher PGAs.

BRIDGES	***DARFIELD 2010****	***CHRISTCHURCH 2011*****
Stanmore Bridge	A	B
Fitzerald Bridge	A	B
Medway Pedestrian Bridge	C	D
Gayhurst Bridge	B	C
Snell Footbridge	C	C
Avondale Bridge	A	B
Anzac Bridge	A	B
South Bridge Bridge	B	B
Ferrymead Bridge	A	B

A	*without visible damage or completely undamaged*
B	*safe bridge for use* ***but*** *with visible damage*
C	*bridge closed due to large but repairable structural and foundation damage*
D	*bridge failed completely*



** from Bridge Research Group (Natural Hazard Platform) inspection (SEPTEMBER 2010)*
*** from our inspection (APRIL 2011)*

Figure 4. Damage assessment of bridges along Avon and Heathcote river due to lateral spreading during Darfield and Christchurch earthquakes. The map illustrates the location of the bridges (red signs) and the seismic stations (green signs) with the corresponding recorded PGAs (**1**st and **2**nd value correspond to Darfield and Christchurch earthquakes respectively) in the broader area occupied by the bridges.

Liquefaction along Avon River caused lateral spreading of the ground. Evidently, bridges were the structures mostly affected by lateral spreading. As Avon river forms several meanders within the city, there is a large number of bridges crossing the river. During our visit in April 2011, we inspected 9 bridges (8 crossing Avon river and 1 crossing Heathcote river). Typically, most of them are small-span concrete bridges with none to two piers, seat type abutments on piles, with the exception of the two pedestrian bridges, i.e. the steel truss Medway bridge and the arch Snell Footbridge. The table in Figure 4 includes all nine bridges and illustrates their response during Darfield and Christchurch earthquakes according to certain performance states described in the legend below the table. It can be clearly concluded that: i) most of the bridges performed very well during Darfield earthquake, apart from Gayhurst and South Bridge Bridges with moderate visible damage and the two pedestrian bridges with significant damage and ii) the performance of the bridges was worse, but still satisfactory in general, during Christchurch earthquake. This is partially attributed to the smaller ground accelerations of Darfield earthquake and thus, to the smaller inertial forces developed to the bridge decks. However, the deflection of the bridges points out lateral spreading as the prevailing source of load.

The deformation pattern of the concrete road bridges consists of lateral displacement of the abutments towards the river and their rotation around their contact point with the deck, accompanied by subsequent settlement of the fill behind the abutment and distress of the piles. This typical pattern of deformation is attributed to lateral spreading of the ground. Aiming to investigate the reason why the bridges that were damaged during the Christchurch earthquake performed really well in Darfield earthquake, despite the fact that liquefaction occurred in both earthquakes, especially in the eastern suburbs, an evaluation of the liquefaction potential with depth was conducted for Anzac Bridge and Fitzgerald Bridge, for both earthquakes (see map of Figure 4).

Following the classical Seed and Idriss (1971) procedure for assessing the liquefaction potential (as updated by Idriss and Boulanger (2006)), the Cyclic Resistance Ratio (CRR) of soil and the applied Cyclic Shear Ratio (CSR) were computed. The former was obtained from the CPT (in case of Anzac Bridge) or SPT data (in the case of Fitzgerald Bridge) and the latter from the PGAs of the corresponding site.

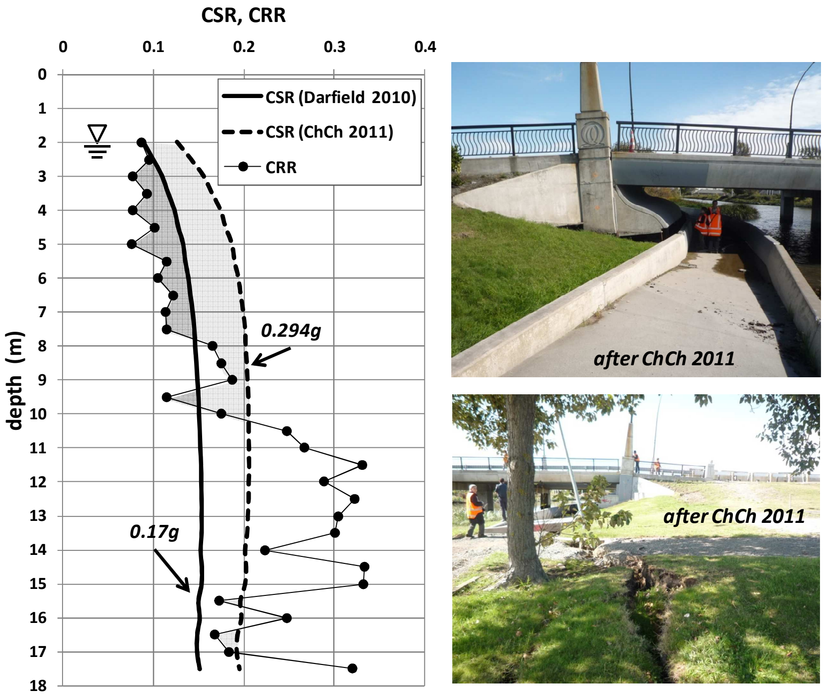


Figure 5. The graph (left) depicts the distribution of CRR with depth of a site between the HPSC station and Anzac Bridge (see map of Figure 4) compared with the CSR one during Darfield and Christchurch earthquakes. The shadowed areas indicate liquefaction susceptibility. The photos (right) illustrate the rotation of the abutments of Anzac Bridge and the cracks due to lateral spreading along the Avon river coast.

The liquefaction potential analysis of the soil profile, obtained from a borehole between the southern abutment of the Anzac Bridge and the HPSC seismic station (located at a distance of 100m approximately) using the PGA records of the aforementioned station (Figure 5), indicates that during the Darfield earthquake (0.17g) the liquefaction took place practically from the ground water level to a depth of 8m (shadowed area). The applied CSR values are larger but close enough to the CRR values, so that liquefaction may not be severe. In the case of Christchurch earthquake (0.29g), the applied CSR is significantly larger than the CRR to a larger depth (10m). These facts imply that liquefaction is certain during Christchurch earthquake and definitely more extensive than the one during Darfield earthquake, causing larger soil lateral movement and at a greater depth – perhaps even below the pile foundation. This justifies partly why the Anzac bridge sustained no damage during Darfield, but 50cm horizontal displacement, plus rotation of abutments during Christchurch earthquake.

In the case of Fitzgerald Bridge the complete soil profile geometry profile (Figure 6a) is available (Bradley et al., 2009). There is an inclination of the third soil layer towards the southern abutment, which tends to cause an additional lateral soil movement at the side of the northern abutment towards East, even below the pile foundation (8m deep), as indicated by the red arrows. This is also supported by the liquefaction potential analysis of the soil profile below the northern and southern abutment (Figure 6b) which indicates that in the soil between 11 and 14m depth the liquefaction susceptibility is greater in the case of the northern abutment. It should be noted that CSR was estimated using an average PGA value of the REHS and CCCC records which are the closest to the bridge site. The southern abutment, shown in Figure 6c, after the Christchurch earthquake, suffered only some moderate cracks, with no significant horizontal displacement or rotation. Moreover, there were no signs of lateral spreading (cracks, gap between abutment and soil) in front of the toe of the abutment. On the other hand, the northern abutment (Figure 6d) sustained substantial rotation, the backfill has settled and the ground in front of the toe has spread towards the river, exposing the top of the piles.

The fact that there was no visible damage of the bridge during Darfield earthquake can be explained by the lower liquefaction potential of the soil profiles (shadowed areas) and its smallest range in depth compared to that obtained for Christchurch earthquake. In addition, there was no ground manifestation of liquefaction close to the bridge site in the case of Darfield earthquake, in contrast to the case of Christchurch earthquake. This observation is also supported by the analysis showing deeper liquefaction (practically below the depth of 8m) during the first earthquake. Thus, it is more difficult for the sand to be ejected on the ground surface.

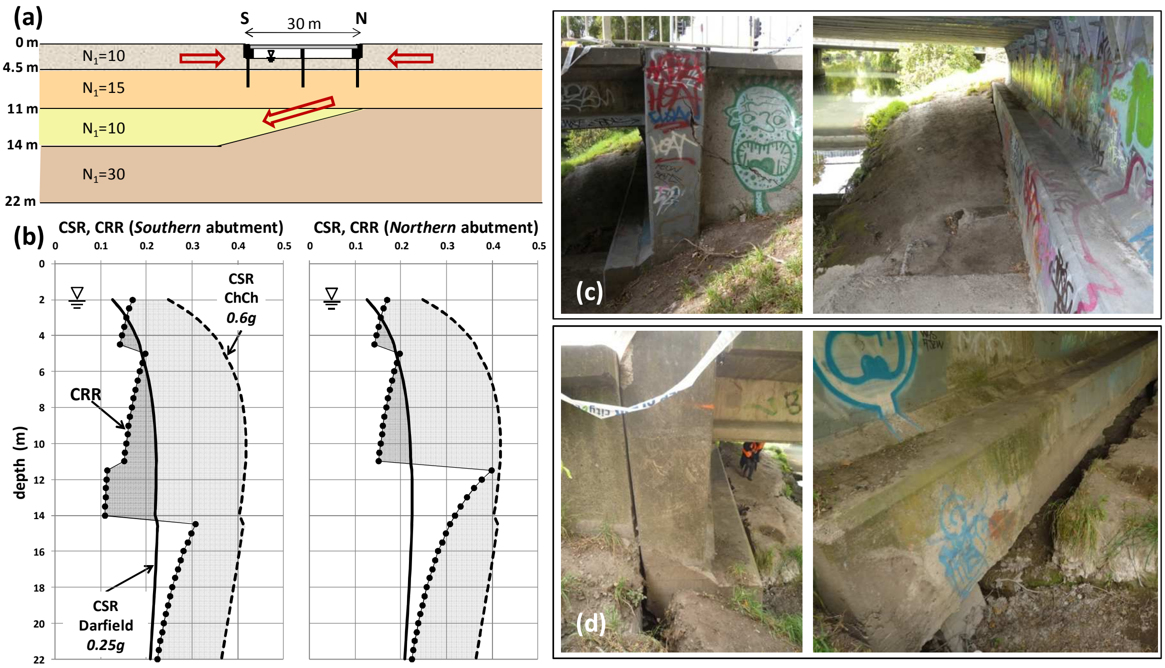


Figure 6. (a) Soil profile geometry along Fitzgerald Bridge. The red arrows indicate the soil displacement due to lateral spreading. (b) Distribution of CRR with depth based on SPT values compared with the CSR for the northern and southern abutments. (c) Performance of southern abutment after ChCh earthquake. (d) Performance of northern abutment after ChCh earthquake.

EFFECTS OF THE EARTHQUAKES ON PARTICULAR STRUCTURAL TYPES

As opposed to the European building stock, the structures in New Zealand exhibit a great variety. Structures built with wood and masonry constitute around 80% of the building stock (Uma at al., 2008). Christchurch in particular has many single storey or 2-storey masonry and timber residential buildings outside the Central Business District (CBD) and very few modern RC high-rise buildings (Figure 7). The building composition in the CBD differs from the rest of the city with medium-rise modern steel and RC structures, as well as mid-rise unreinforced masonry (URM) and timber dwellings and office buildings, some of which have historical value.

It can be said, very roughly, that the Christchurch City, outside of CBD, consists of relatively low-period structures, whilst the long-period structures are more common in CBD. This might be one of the reasons of the high concentration of damages in the CBD area during the February 2011 earthquake. A preliminary study presented in this paper investigates the spatial distribution of the spectral amplifications of the recorded strong motions for several yield period ranges as explained below, noting that the period ranges examined have been chosen in accordance with the observed building characteristics of the region.

Figure 7. Characteristic timber (left), masonry (middle) and modern RC (right) structures outside of CBD.

Damage to Building Structures during the Darfield and Christchurch Earthquakes

Darfield earthquake inflicted severe damage to residential houses and infrastructure mainly due to soil liquefaction and lateral spreading. The damage was concentrated in areas close to major streams, rivers and wetlands throughout Christchurch and the town of Kaiapoi (Cubrinowski et al., 2010). All buildings were assigned a usability rating of green, yellow or red tag. Green indicted no limitation of access and usability, yellow signified restricted use only, while red meant that a building was unsafe and access was banned. According to the Civil Defence Council, 80% of the buildings inspected were tagged green, 14% yellow and 6% red, while no building collapse was reported. As for the unreinforced masonry structures, a preliminary assessment on approximately 600 such buildings (Ingham and Griffith, 2010) resulted in 21% of the URM structures to be assigned red, 32% yellow and 47% green tag.

As for the Christchurch earthquake, at the time of writing this paper the final statistics regarding the building safety evaluation were not yet available. However, as per 18^{th} of March, the data by Civil Defence (Kam et al., 2011) referred to 3621 buildings checked within CBD, out of which 1933, 862 and 826 were posted red, yellow and green respectively. Being more specific, of the “red” buildings 19% were reinforced concrete structures, 14% timber structures and only 7% the steel buildings, which performed in general satisfactorily. The respective percentage for reinforced masonry structures was 16%, jumping to 62% for unreinforced masonry buildings, reconfirming the poor behaviour of such structures, most of which have been built following the dominant construction practice or earlier design codes. Insufficient detailing and bad construction techniques, mostly related to non-structural elements, deteriorated the damage. Although the aforementioned data have come up before the completion of the 2^{nd} level of building safety assessment and thus reflect the situation in CBD one month after the earthquake, they offer a representative picture of the extent and severity of damage in CBD

Correlation of Damage to Spectral Values

Estimation of Yield Period of Characteristic Structural Types

The yield period of a building refers to the stiffness at the point of yielding, which essentially signifies the limit beyond which the structure enters inelasticity and starts to experience substantial damage that may require structural repair. A schematic representation of the characteristic limit states and the relevant bi-linearization of the system are given in Figure 8. A multi-degree-of-freedom (MDOF) system can be represented by an equivalent single-degree-of-freedom system (SDOF) precisely enough, as first suggested by Gülkan and Sözen (1974) and Shibata and Sözen (1976), as long as the equivalent SDOF substitute structure’s characteristics are determined. An effort to define the yield period that corresponds to the K_{LS1} is made in this study aiming to have an overall idea about the spatial distribution of the damage.

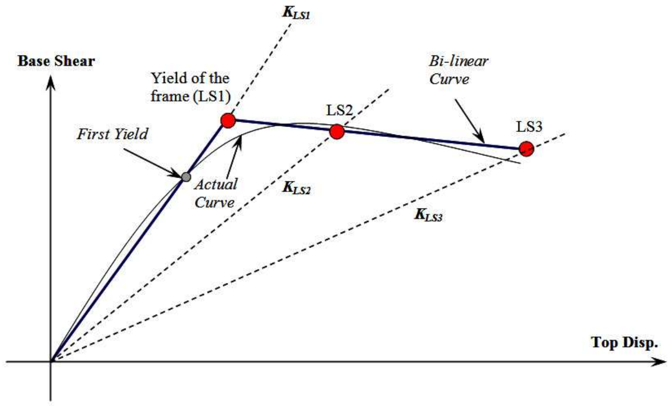


Figure 8. Structural response in base shear vs top displacement format, structural limit states and the bi-linearization process (by Bal et al., 2007).

Due to the space limitations, only two structural types have been considered in the preliminary analyses presented. The first group is the typical mid-rise RC structures in the city, most of which have been built with RC walls around stairs and elevator shafts (Uma et al., 2008), a situation also observed in European building stock (Bal et al., 2007). Studies by Vuran et al. (2008) and Enrique (2010) on such structures, where RC walls have not been built with the primary concern of earthquake resistance, but in reality, nevertheless, they contribute to the seismic response, show that a period value as given in Equation (1) would be a fairly good approximation to the yield period of similar structures.

$$T_y \approx 0.075H \tag{1}$$

where H is the total height in m. The yield period of a 6-storey representative structure with RC walls around the elevator shaft then becomes 1.36sec.

As described above, the majority of residential buildings in Christchurch consist of 1 to 2-storey unreinforced masonry buildings with timber floors and ceiling, thus, this group was also examined. A study by Bothara et al. (2007) on a representative laboratory structure suggests that the threshold drift limits describing slight, moderate, extensive and complete damage states are 0.1, 0.4, 0.9 and 1.3% respectively. The elastic period found in their study is 0.09sec in average for the two transversal directions. This elastic period, refers to the stiffness that corresponds to the drift limit of 0.1%, which defines the limit for the slight damage. If real-size structures with two floors and 2.8m storey height are considered, in conjunction with an assumed value of 0.6 (ATC, 1996) for the ratio of the force where the first crack occurs over the yield force, then the yield period can be calculated as 0.26sec.

Distribution of Spectral Values

The spectral values of acceleration and displacement for the two consecutive earthquakes have been computed and plotted in the contour maps shown below. Only the maximum of the EW and NS components are presented here due to space restrictions. A range of spectral values within a band of ±10% has been averaged in order to produce the following plots, noting that the values have been calculated per station and linearly interpolated among the stations, thus, the plots should be considered more indicative rather than definitive.

Spectral acceleration and spectral displacement plots given in Figure 9 indicate that for the short period range where the low-rise timber and URM buildings fall into, the concentration of spectral amplifications occurs near the western end of the fault. This is somehow interesting since Figure 2 reveals a concentration of PGAs along the fault rupture line, implying that the motion right along the fault may contain mostly long period components. Another observation derived from Figure 9 is the displacement demand for the SDOF systems. Uma et al. (2008) claims that the drift limit state for URM buildings for the moderate damage is 0.4%, which is translated to a

0.012m displacement demand for an equivalent SDOF system, explaining thus the damage concentration of such structures in the Darfield area after the September'10 earthquake.

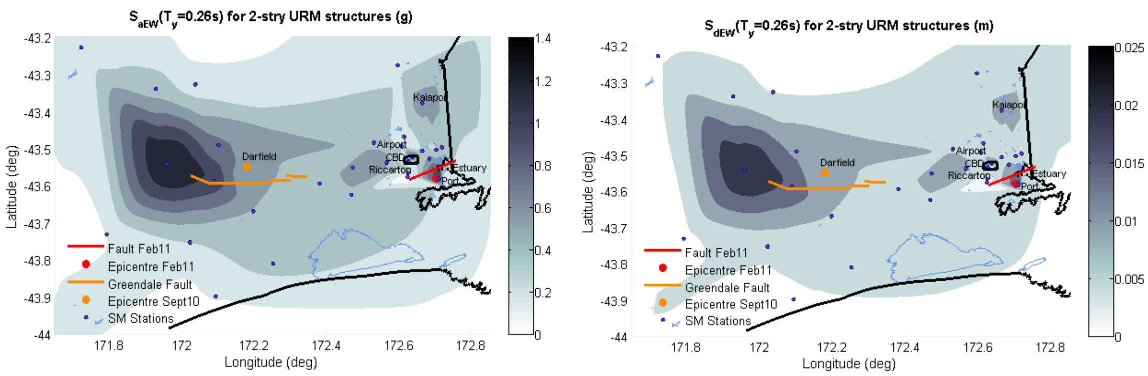


Figure 9. Spatial distribution of spectral accelerations (left) and displacements (right) for Darfield earthquake for yield period of 0.26sec (2-storey typical URM houses-EW direction).

An interesting feature in Figure 10, in relation to the comments given above on Figure 9, is that the long period spectral amplifications appear more pronounced on the western end fault, a probable outcome of forward rupture directivity effects that affect the frequency content of the strong motion records with respect to the position and the site distance from the fault (Somerville, 2003). The spectral acceleration values in Figure 10 highlight the very high acceleration demands imposed (in the range of one g), but, as the building exposure in that region is mostly low-rise residential, extensive damage of mid-rise RC structures was not observed after the September'10 earthquake.

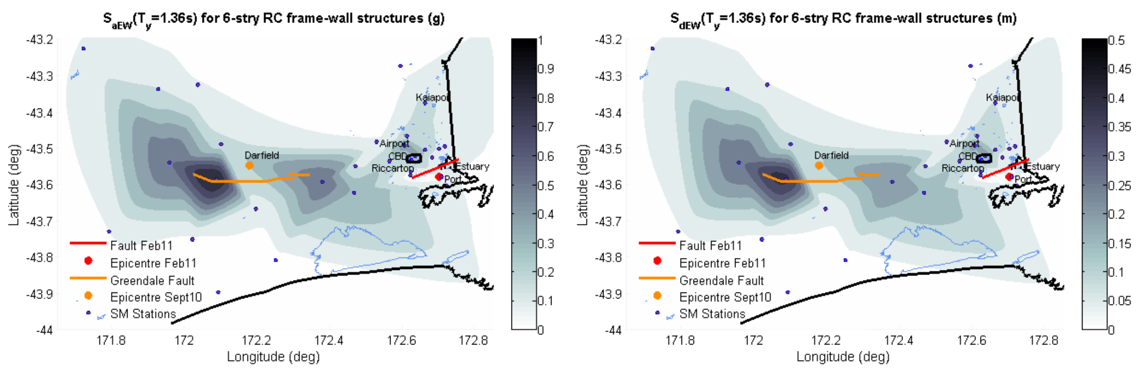


Figure 10. Spatial distribution of spectral accelerations (left) and displacements (right) for Darfield earthquake for yield period of 1.36sec (6-storey typical RC frame-wall structures - EW direction).

The spatial distribution of PGAs and long-period spectral values exhibit a gradual reduction of values with increasing distance from the source, as readily shown by the contour lines, referring to a point-source for the February'11 earthquake rather than a line source as happened in the September'10 earthquake. The seismological definitions concerning the Christchurch earthquake are yet not settled, due to the fact that there was no surface rupture, and even the debate on whether the February'11 earthquake belonged to the aftershock sequence of the Darfield earthquake or not is still open. Nevertheless, from an engineering point of view it is rather interesting noticing in Figure 11 that the short-period components are concentrated in Heathcote Valley, where the epicentre of the earthquake was, while Figure 12 suggests that the long period components are stronger close to CBD, 7km away from the epicentre. This finding can correlate the damage expected to certain structural types in specific areas to the damage really observed.

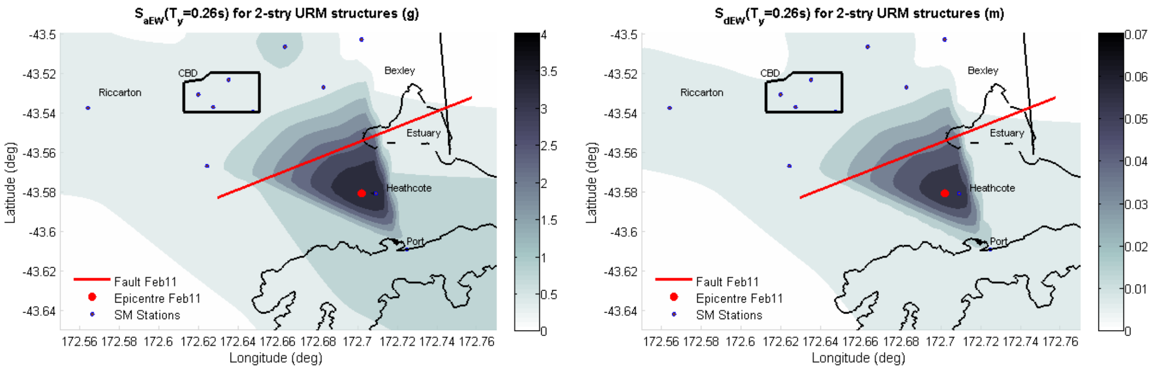


Figure 11. Spatial distribution of spectral accelerations (left) and displacements (right) from Christchurch earthquake for the yield period of 0.26sec (2-storey typical URM houses - EW direction).

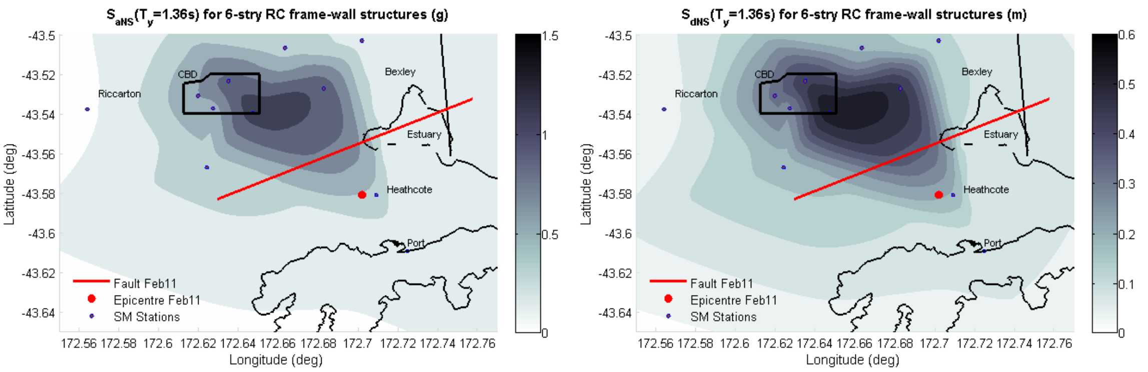


Figure 12. Spatial distribution of spectral accelerations (left) and displacements (right) for Christchurch earthquake for the yield period of 1.36sec (6-storey typical RC frame-wall structures - NS direction).

As mentioned, the life losses from the second earthquake were primarily due to the heavy damage or collapse (mostly partial) of mid-rise and relatively tall RC structures in the CBD. Indicatively, the case of Grand Chancellor Hotel, which is a 26-storey structure with severe damages and more than one meter residual top displacement, is mentioned as an example of a tall RC building with substantial structural damage that rendered it unusable. Figure 12 presents this issue quite clearly showing that the spectral amplification around CBD is much higher than the one close to the fault epicentre. Nevertheless, the effects of the Christchurch earthquake on tall structures in CBD needs to be investigated in detail with respect to the soil-structure interaction and the soil plastification recorded in several parts of the centre.

Finally, it is worth noticing the different composition of the spectral values for short and long periods in different regions in respect to the fault (Figure 9 to Figure 12), which could be partly attributed to potential source effects. However, another plausible explanation is the soil softening due in particular to soil liquefaction that occurred in large scale in both earthquakes. Response spectra obtained in liquefied areas are often characterised by bulges in the long-period range, signifying substantial amplification of the spectral values, much further than the constant acceleration plateau (Figure 12). Such a change in the frequency content, evident also in the spatial plots, has a significant impact on the seismic demand imposed and thus on the response of mid-rise and tall structures.

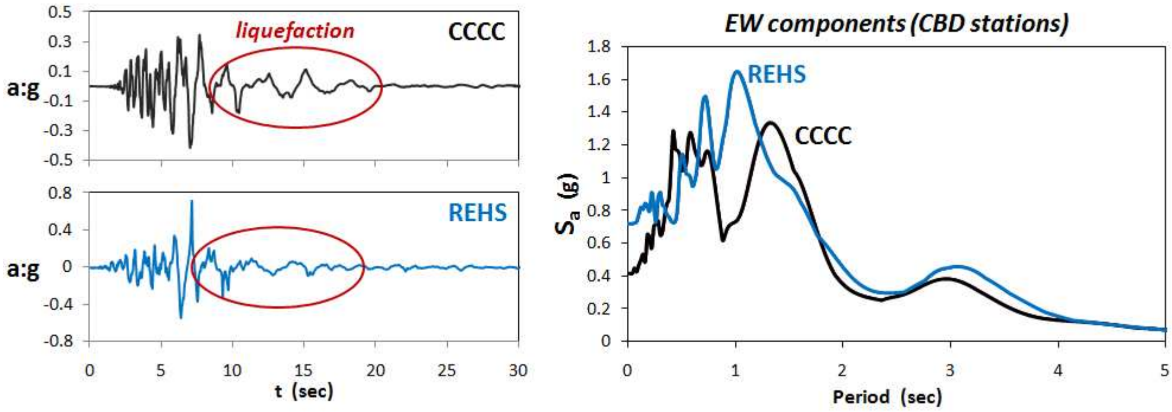


Figure 12. Recorded acceleration time-histories (left) and response spectra from REHS and CCCC stations (right) indicative of liquefaction.

CONCLUSIONS

The city of Christchurch, New Zealand, belonged to a high seismicity zone, however, the two recent earthquakes of September 2010 and February 2011, generated on unknown faults, created ground motions well above the design spectra. Apart from the new seismological data revealed after Darfield and Christchurch earthquakes, several interesting aspects in terms of soil response and performance of structures have also been noted by engineers. Both earthquakes caused severe soil liquefaction and re-liquefaction throughout the city and the surrounding areas. In particular, the Christchurch earthquake was especially destructive for the city centre. Severe damage and/or collapse was concentrated on URM structures and taller RC buildings. This paper examined the reasons that led to extensive structural damage of buildings by trying to relate the dynamic behaviour of characteristic types of structures to the spectral demands. Furthermore, evaluation of liquefaction potential was conducted in order to assess the performance of bridges due to lateral spreading. It was also shown that the effect of soil liquefaction on long-period structures and the near-source effects on buildings were critical for their response and the associated level of damage. Further research, supported by the continuously increasing amount of data, will offer better insight into the structural and geotechnical aspects of these two earthquake events.

Acknowledgements

The authors would like to thank Professors John Berrill, Misko Cubrinovski, Stefano Pampanin and Dr. Umut Akguzel for providing data and assisting the authors during their visit in Christchurch in April 2011. The financial support for this paper has been provided under the research project “DARE”, which is funded through the European Research Council’s (ERC) “IDEAS” Programme, in Support of Frontier Research-Advanced Grant, under contact/number ERC-2-9-AdG228254-DARE.

REFERENCES

ATC-40 (1996) Seismic Evaluation and Retrofit of Concrete Buildings, Report No. ATC-402, Applied Technology Council, Redwood City, California.

Bal, İE, Crowley, H, Pinho, R, Gülay, G (2007) Structural characteristics of Turkish building stock in Northern Marmara region for loss assessment applications, ROSE Research Report 2007/03, IUSS Press, Pavia, Italy.

Blair E. (2010) Equivalent linearization of the existing 3D frame-wall structures for displacement-based assessment, MSc Thesis, IUSS Rose School, Pavia, Italy.

Bothara, JK, Mander JB, Dhakal, RP, Klare, RK, Maniyar, MM (2007) “Seismic performance and financial risk of masonry houses”, *ISET Journal of Earthquake Technology*, 44(3-4):421-444.

Bradley, BA, Cubrinovski, M, Dhakal, RP, MacRae, GA (2009) "Probabilistic Seismic Performance Assessment of a Bridge-Foundation-Soil System", *Proceedings of the NZSEE Conference*, Christchurch, N. Zealand, 3-5 April.

Crowley, H and Pinho, R (2004) "Period-height relationship for existing European reinforced concrete buildings", *Journal of Earthquake Engineering*, 8(S1):305-332.

Cubrinovski, M, Green, R, Allen, J, Ashford, S, Bowman, E, Bradley, BA, Cox, B, Hutchinson, T, Kavazanjian, E, Orense, R, Pender, M, Wotherspoon, L (2010) Geotechnical reconnaissance of the 2010 Darfield (New Zealand) earthquake. Report, University of Canterbury, Christchurch, N.Zealand.

Gledhill, K, Ristau, J, Reyners, M, Fry, B, Holden, C, (2011) "The Darfield (Canterbury, New Zealand) Mw 7.1 Earthquake of September 2010: A Preliminary Seismological Report", *Seismological Research Letters*, 82(3):378-386.

Gülkan, P and Sözen, M (1974) "Inelastic response of reinforced concrete structures to earthquake motions", *ACI Journal,* 71 (12): 604-610.

Idriss, IM, Boulanger, RW (2006) "Semi-empirical procedures for evaluating liquefaction potential during earthquake", *Journal of Soil Dynamics and Earthquake Engineering*, 26:115-130.

Ingham, J and Griffith, M (2010) Performance of unreinforced masonry buildings during the 2010 Darfield (Christchurch, NZ) earthquake. Report, NZSEE Library, N.Zealand.

Kam, WY, Akguzel, U, Pampanin, S (2011) 4 Weeks on: Preliminary reconnaissance report from the Christchurch 22 Feb 2011 6.3Mw Earthquake. Report, NZSEE Library, N.Zealand.

Seed, HB, Idriss, IM (1971) "Simplified procedure for evaluating soil liquefactio potential", *Journal of the Soil Mechanics and Foundations Division*, ASCE, 97(SM9):1249-73.

Shibata, A, and Sözen, M (1976) "Substitute structure method for seismic design in reinforced concrete", *ASCE Journal of the Structural Division*, 102(ST1):1-8.

Somerville, PG (2003) "Magnitude scaling of the near fault rupture directivity pulse", Physics of the Earth and Planetary Interiors, 137:201-212.

Tonkin & Taylor Ltd (2011) Geotechnical factual reports. available at http://canterbury.eqc.govt.nz

Uma, SR, Bothara, J, Jury, R, King, A. (2008) "Performance Assessment of Existing Building in New Zealand", *Proceedings of the NZSEE Conference*, Wairakei, N.Zealand, 11-13 April, 45.

Vuran, E, Bal, İE, Crowley, H, Pinho, R (2008) "Determination of Equivalent SDOF Characteristics of 3D Dual Structures", *Proceedings of the 14WCEE,* Beijing, China, 8-12 October, S15-031.