THE M_W 6.3 CHRISTCHURCH, NEW ZEALAND EARTHQUAKE OF 22 FEBRUARY 2011

A FIELD REPORT BY EEFIT

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1. INTRODUCTION

THE CHRISTCHURCH EARTHQUAKES

At 04:35 (local time) on Saturday 4th September 2010, the province of Canterbury experienced a moment magnitude (M_w) 7.1 earthquake. The epicentre of this earthquake was approximately 40 km west of Christchurch near the town of Darfield. The earthquake was noteable for the widespread liquefaction it caused and for the large amount of damage in the city or Christchurch and particularly its surrounding areas. However, as no modern engineered structures suffered significant structural damage and no lives were lost (about 150 people were injured), this earthquake was not particularly significant in terms of global seismic events.

As is usual with seismic events of this magnitude, the region experienced an aftershock sequence; the most significant occurring on the 26th December, 2010, and the 22nd of February, 2011. The close proximity of the epicentre to Christchurch and the shallow focus of the latter of these two aftershocks resulted in widespread structural damage, collapse of buildings, disruption to services and the loss of 182 lives. While the magnitude of this earthquake was not particularly large, the earthquake was significant for a number of reasons. From a seismological point of view it was significant for its high ground accelerations especially in the vertical direction and from an engineering point of view, this earthquake is important as it was the first earthquake since the 1999 Chi Chi event that has occurred in a developed region with not only modern building codes, but a good construction industry. The final important feature of note for this earthquake is that the ground accelerations were much greater than the design ground accelerations and so this event presents a good opportunity to assess the life-safety and collapse performance limits of the modern, seismically designed structures in this region.

This report presents the observations and findings of a reconnaissance mission to Christchurch and its surrounding areas made by the Earthquake Engineering Field Investigation Team (EEFIT). Its purpose was to identify new lessons in seismic design that can be learnt from the event and these are presented in the following report.

THE MISSION

On 25th February the EEFIT management committee held a meeting and decided to launch a mission to Christchurch and the area affected by the 22nd February earthquake. With this purpose Dr Matthew Free was selected as team leader for the mission. The other team members were selected to cover a wide range of expertises including engineering seismology, engineering geology, earthquake engineering, structural engineering, seismic risk analysis, risk modelling, lifelines and geotechnical earthquake engineering. The team members are the authors of this report and are shown in Figure 1. The team departed for Christchurch on the 11th of March and spent 5 days in the field collecting data.

Figure 1 – EEFIT team members. From left to right: front row – Anna Mason, David Boon; back row – Stuart Fraser, Damian Grant, Matthew Free, Sara Paganoni, Elizabeth Williams, Sean Wilkinson.

2. REGIONAL TECTONIC AND GEOLOGIC SETTING

TECTONICS AND HISTORICAL SEISMICITY

New Zealand is located on the tectonic plate boundary between the Australian Plate to the west and the Pacific Plate to the east (see Figure 2). In the very south of the country, in the Fiordland region, the Australian Plate is subducting obliquely to the east beneath the Pacific Plate at the Puyseguer Trench. To the east of the North Island of New Zealand, the Pacific Plate is subducting obliquely to the northwest beneath the Australian Plate at the Hikurangi Trough.

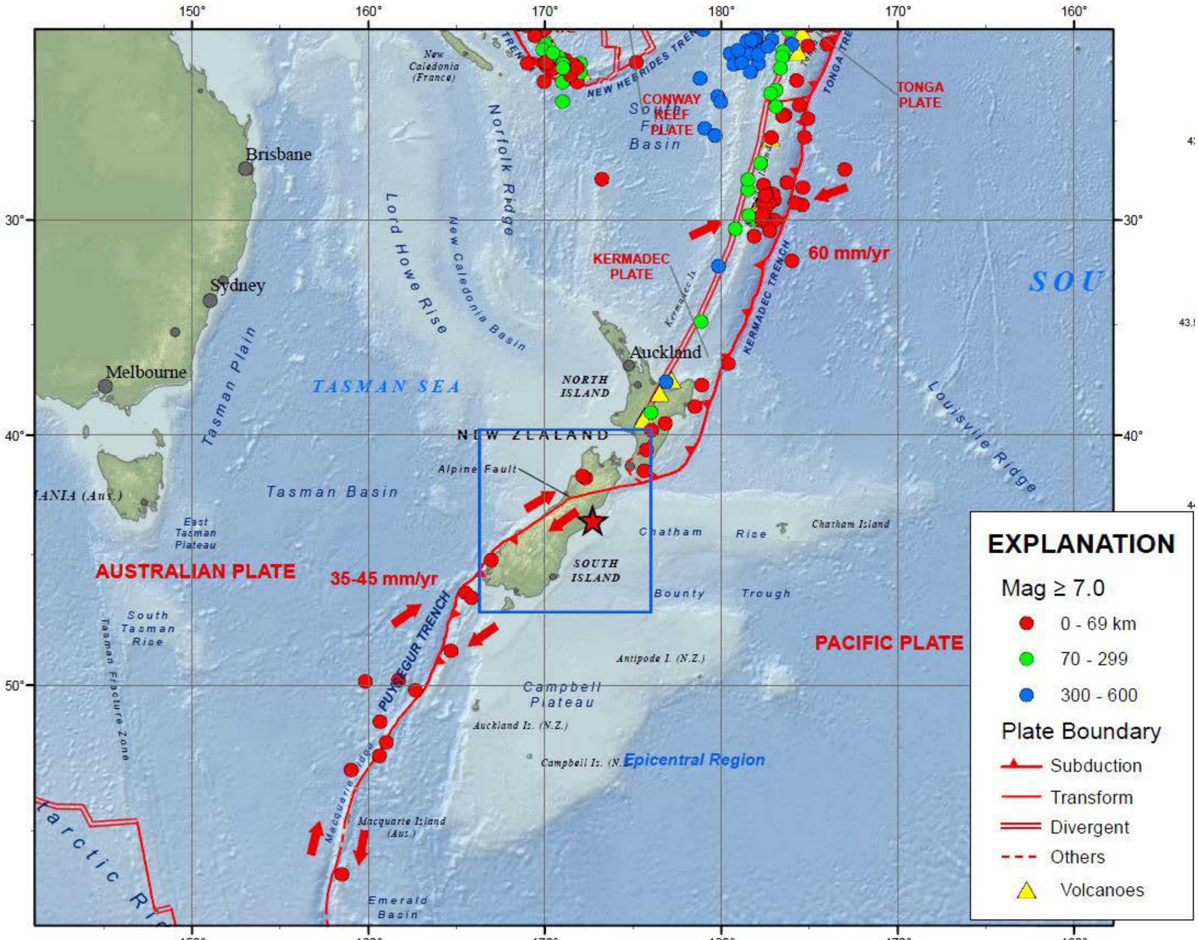


Figure 2 Tectonic setting of New Zealand (USGS, 2010)

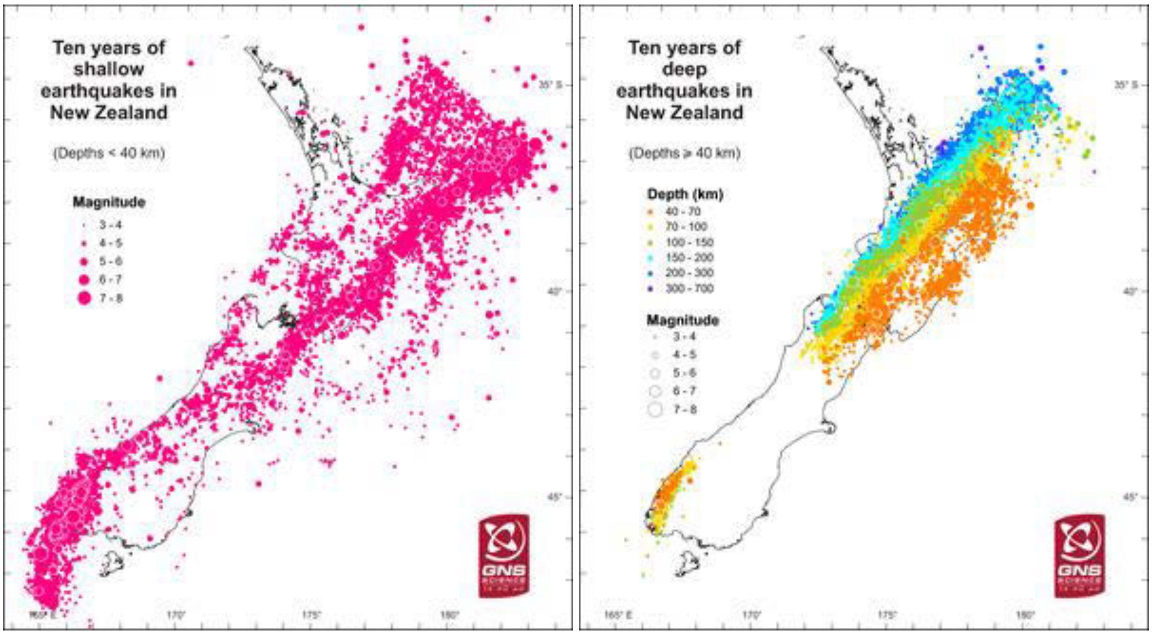
Between these two subduction zones, the plate boundary is characterised by a zone of right lateral strike slip faulting and oblique continental collision that extends for almost 700 km from the south-west corner to the northeast corner of the South Island of New Zealand. In the central South Island, the relative velocity between the Pacific Plate and the Australian Plate is approximately 40 mm/year. The majority of the relative displacement between the tectonic plates, approximately 75%, is taken up on the Alpine Fault, approximately 20%

distributed on faults within the Southern Alps and the remaining 5% (approximately) on faults within a broader region beneath the Canterbury Plains (Wallace et al., 2007). Geological and seismological studies indicate that approximately 75% of the motion is strike-slip and 25% dip-slip. The horizontal (strike-slip) component of displacement has resulted in 480 km of cumulative displacement since the Late Oligocene-Early Miocene (i.e. over a period of about 15 million years) implying an average slip rate of about 32 mm/year. The vertical (dip-slip) component of displacement is largely responsible for the uplift of the Southern Alps (DeMets et al., 2010; Sutherland et al., 2006; Wallace et al., 2007). In the central South Island the dominant tectonic feature appears to be the Alpine Fault although there are a number of additional mapped active faults within the Southern Alps (Stirling et al., 2008). No active faults are mapped beneath the Canterbury Plains but seismo-tectonic models for the region indicate strike-slip and reverse faults are present and modelled as diffuse zones (Stirling et al., 2008).

As a result of this tectonic setting, New Zealand is a region of high seismicity, with the majority of seismicity occurring in broad zones associated with the tectonic features described above. Well defined zones of deep seismicity are associated with the subduction zones to the south and north of the South Island. More diffuse, shallow seismicity occurs in the central region of the South Island. Figure 3a shows the shallow (less than 40km depth) earthquakes and Figure 3b shows the deeper earthquakes with colours indicating depth. The various depths of the earthquakes are related to the subduction zones in the north and the south and only shallow earthquakes occur along the Alpine fault and in the Southern Alps of New Zealand and along the Canterbury Plain. Paleoseismic and historical evidence suggests that the Alpine Fault ruptures in major earthquakes (M_w > 7.5) with recurrence intervals of approximately 200 to 300 years, with the most recent major event in 1717 (Sutherland et al., 2006). A number of large earthquakes (6 < M_w < 7) have occurred in the Southern Alps over the last 150 years. These include: the 1888 North Canterbury M_w = 7.1 event; the 1929 Arthur's Pass M_w = 7.0 event; the 1994 Arthur's Pass M_w = 6.7 event; and the 1995 Cass M_w = 6.2 event (GeoNet, 2011). The Canterbury Plains are a region of lower seismicity with fewer large earthquakes but numerous small to moderate size earthquakes. There is no historical record of a large earthquake in the immediate region of Darfield. However, it should be noted that the written historical record in New Zealand is relatively short at about 170 years. Little is known of the pre-European earthquakes as there is no written record. A longer pre-history of earthquakes is preserved in Quaternary age fault scarps some of which show repeated fault ruptures (Pettinga et al. 2001; Howard et al. 2005).

Forsyth et al. (2008) report a series of historical earthquakes in the Christchurch region. In June 1869, an earthquake with shaking intensities up to MM7 or MM8 brought down chimneys and damaged masonry buildings in Christchurch. The earthquake is interpreted to have had a shallow focus beneath Christchurch city and may have been up to magnitude M_w = 5.8. Lower magnitudes in the range M_w = 4.7 to 4.9 are also reported. In August 1870 an earthquake caused further chimney damage in Christchurch and the nearby port of Littleton. Magnitude values in the range M_w = 5.8 to 6.5 are assigned for the event based upon felt intensities. In September 1971 a shallow event with magnitude M_w = 4.5 caused felt intensities of MM5 in Christchurch. In March 1987 a magnitude M_w = 5.2 earthquake occurred offshore beneath Pegasus Bay about 60 km northeast of Christchurch.

Forsyth et al. (2008) concluded with considerable foresight that, "future moderate to large earthquake can be expected on active faults in the Christchurch map area, resulting in local damage. Large earthquakes may also occur on undetected faults that do not extend to the ground surface …"



a) Shallow focus b) deep focus

Figure 3 Distribution of recent historical seismicity in New Zealand (GNS, 2011)

REGIONAL GEOLOGY AND GEOMORPHOLOGY

The geology and geomorphology of the region are largely the result of the changing dynamics of the Australian – Pacific tectonic plate boundary during the Neogene period (over the last 25 million years).

The basement rocks are predominantly Mesozoic age (250 to 65 million years) sedimentary and metamorphic rocks belonging to the Torlesse composite terrane that were originally part of the Gonwanaland supercontinent (Forsyth et al., 2008). This terrane comprises mainly indurated sandstone and mudstone, informally known as greywacke.

Volcanic episodes have occurred in the region since the end of the Cretaceous period (65 million years); one of these, in the Miocene (23 to 5 million years) formed the Banks Peninsula (Forsyth et al., 2008). Banks Peninsula comprises the deeply eroded remnants of two large and overlapping volcanoes consisting predominantly of alternating lava flows and scoria deposits. Lava flows on the peninsula have been dated at 6 to 7 million years before present. The highest point on Banks Peninsula is Mt Herbert at 919 m.

During the more recent Quaternary (2.5 million years to present), the region has been affected by both ongoing active tectonics and by glacial processes. Uplift and glaciations in the mountains was associated with widespread sedimentation on the Canterbury Plains and coastal lowlands, resulting in thick fluvial sedimentary deposits beneath the plains and coastal zone. Coastal progradation of up to 12 km has occurred over the last 4000 years (Forsyth et al., 2008) with the formation sand spit and dune deposits. Post-glacial sea-level rise has inundated the coast in some areas, resulting in the formation of estuaries and swamps such as the Avon-Heathcote Estuary upon which the city of Christchurch has been built.

A geology map of the Christchurch area is reproduced from Geology of Banks Peninsula 1:100,000 scale geological map (Sewell et al., 1993) in Figure 4. A geological cross-section through the city is shown in Figure 5. The greater Christchurch area is underlain by predominantly recent Holocene alluvial gravel, sand and silt of the Springston Formation, with Christchurch Formation sediments mapped along the eastern part of the city. The Riccarton Gravel Formation underlies the Springston and Christchurch Formations. Bedrock is at a depth of approximately 600 m to 800 m. The Springston Formation alluvial deposits can be divided into overbank deposits of sand and silt and river flood channels that contain alluvial gravel as the main component. The overbank deposits of sand and silt are expected to be the materials most susceptible to liquefaction. The Christchurch Formation units are described as fixed and semi fixed dunes, and beach sands and therefore are expected to be less susceptible to liquefaction.

The groundwater table affecting the nominal upper 10 m to 20 m of sediments is generally between 2 m to 3 m below the ground surface in the west of Christchurch and 0 m to 2 m below the ground surface in the central and eastern areas of Christchurch.

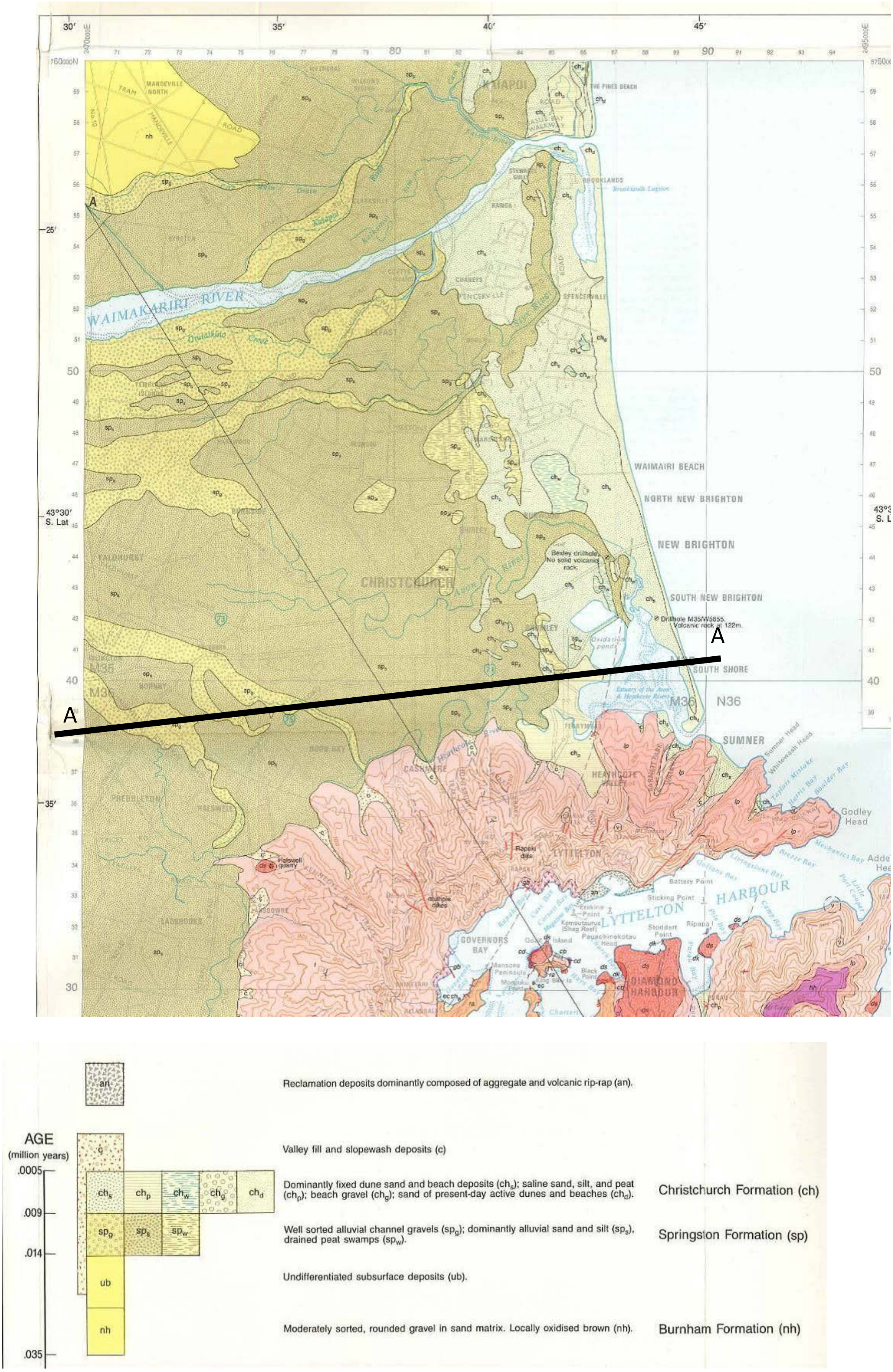


Figure 4 Geological Map of the Christchurch Area (Sewell et al., 1993)

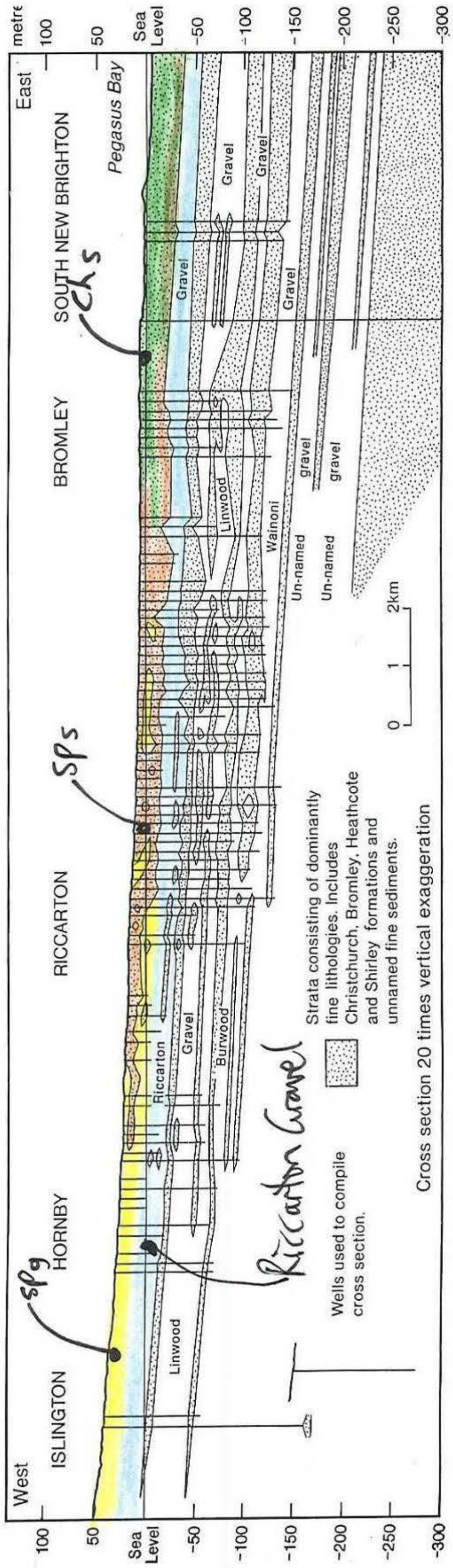


Figure 5 Geological cross-section A-A through Christchurch (from Brown & Webber, 1992). The approximate alignment is shown on Figure 4.

3. SEISMOLOGICAL ASPECTS

THE DARFIELD AND CHISTCHURCH EARTHQUAKES

The key seismological features of the Darfield earthquake are summarised in Table 1 and the location shown in Figure 6 (Darfield earthquake epicentre is shown by a green star). The earthquake occurred at 04:35 on Saturday 4th September 2010 local time (16:35 on 3rd September 2010 UT) with a moment magnitude of M_w = 7.1. The earthquake was located about 10 km southeast of the town of Darfield and 40 km west of the city of Christchurch. The event is reported to have had a focal depth of 8 to 10 km.

Table 1 Key seismological features of the Darfield earthquake

Origin Time	4th September, 2010 at 04:35 NZST (3rd September 2010 at 16:35 UT)
Epicentre	43.55 South, 172.17 East
Depth	8 km (GeoNet), 10 km (USGS)
Magnitude	M_w = 7.1
Focal mechanism	Oblique right lateral strike-slip and reverse on oriented E-W, subvertical fault plan with complex fault rupture on several fault segments
Fault rupture	30km long surface rupture with average horizontal displacement of 2.5m, 5m maximum and a maximum vertical displacement of 1.5m.

The earthquake ruptured the previously unknown Greendale Fault. The location and alignment of the surface rupture is shown by a red line on Figure 6. The surface rupture associated with this event was approximately oriented East-West and movement was predominantly right lateral strike-slip with an average horizontal displacement of 2.5m (maximum 5m) and vertical offset of 1.5m. The surface rupture occurred in a region of relatively flat farmland with very few buildings. However, the fault rupture did cross a number of country roads, a railway and in close proximity to an electrical sub-station and a small number of farm buildings. Photographs of the surface rupture are shown in Figure 7.

The Darfield earthquake was followed by an energetic and lengthy sequence of aftershocks with 4000 events in the first 4 months and strongly felt continuing almost 12 months after the September 2010 mainshock. The distribution of aftershocks up to mid-June 2011 is shown in Figure 6.

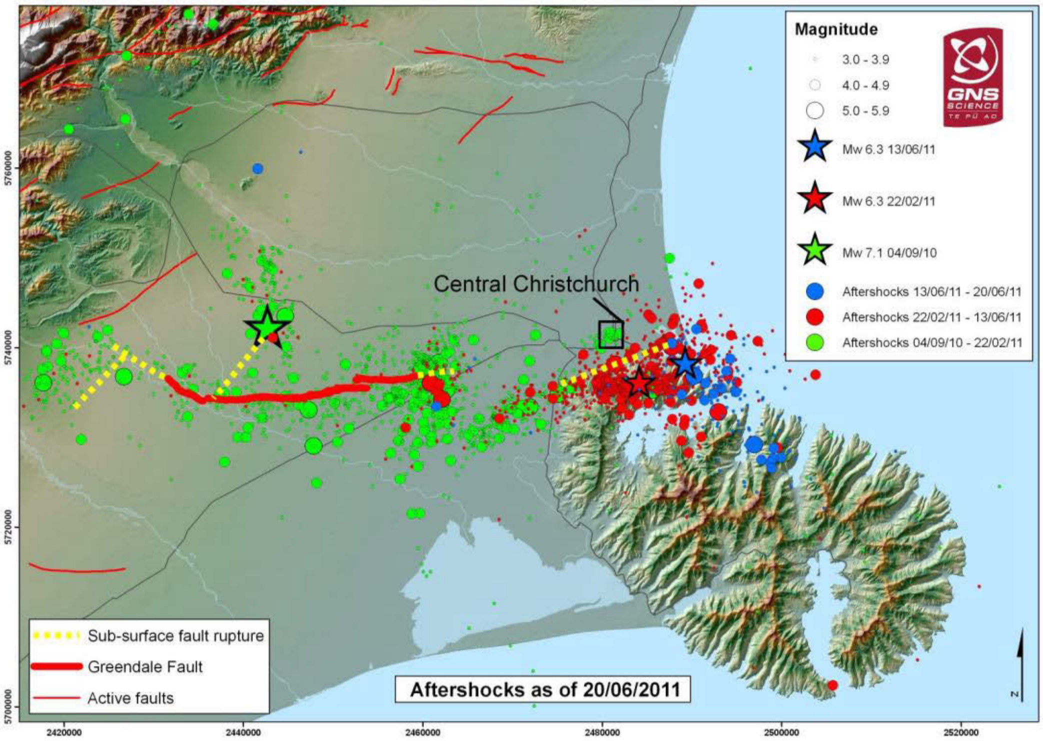


Figure 6 Map showing the locations of Darfield mainshock and the Christchurch aftershock and the smaller aftershock sequence epicentres (GNS, 2011)

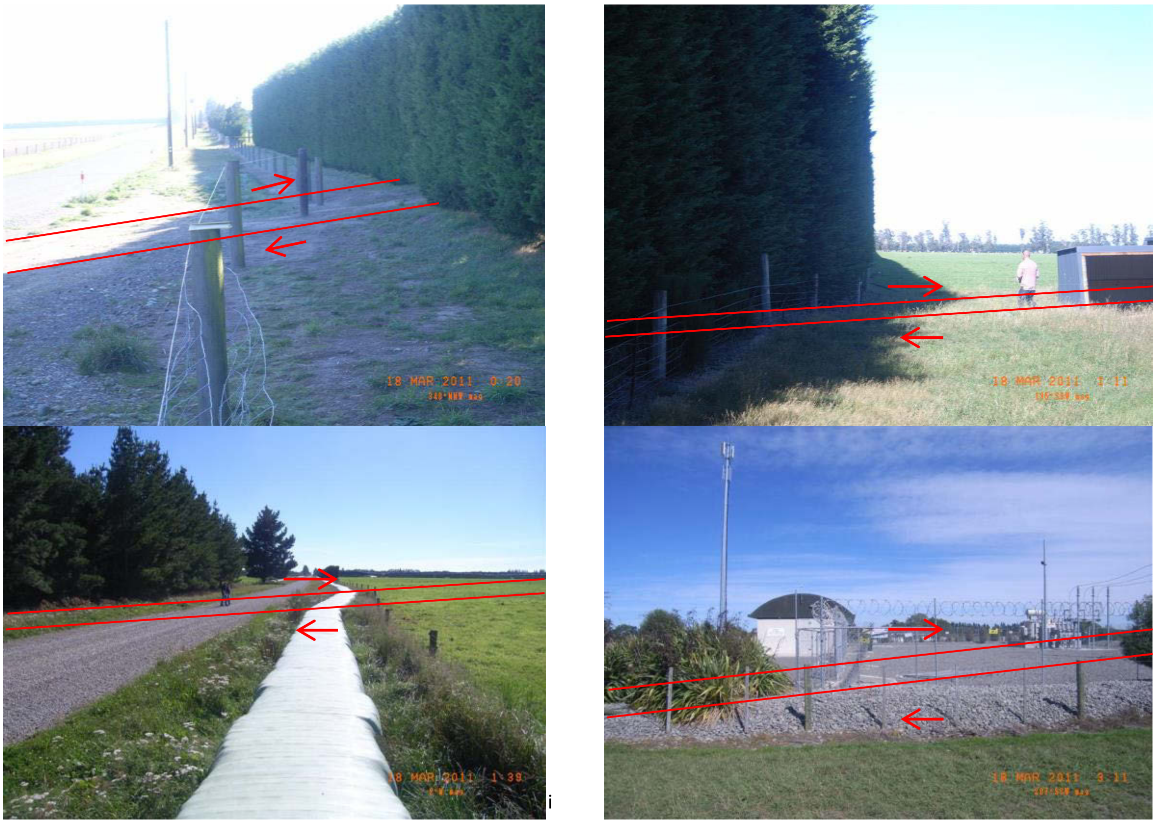


Figure 7 Photographs of right lateral strike-slip displacement offset of fence lines and other features caused by the Darfield earthquake measured in March 2011, about 5 months after the earthquake

At 12.51 on Tuesday 22 February 2011 (local time) a local magnitude ML = 6.3, moment magnitude M_w = 6.2, aftershock occurred with an epicentre located less than 10 km southeast of downtown Christchurch (the epicentre is shown by a red star on Figure 6). The event is reported to have had a shallow focal depth of 5 to 6 km. The key seismological features of the Christchurch earthquake are summarised in Table 2.

Table 2 Key seismological features of the Christchurch earthquake (aftershock)

Origin Time	22 February 2011 at 12:51 NZST (21 February 2011 at 23:51 UT)
Epicentre	43.5834°S 172.7012°E
Depth	5 km (GeoNet), 6 km (USGS)
Magnitude	M_L =6.3, M_w = 6.2
Focal mechanism	Reverse mechanism on ENE-WSW oriented, steeply south dipping fault
Fault rupture	No surface rupture, blind thrust fault, length 10 to 15 km, vertical displacement 0.8 m (average) and 1.5 m (maximum)

The fault rupture that generated the Christchurch event occurred on a previously unknown blind reverse fault, i.e. there was no surface rupture. The fault, located beneath the southern edge of the Avon-Heathcote Estuary, is oriented ENE-WSW and dipping towards the south at about 65 degrees and to have length in the order of 8 to 10 km. The distribution of the permanent ground displacement associated with the Christchurch earthquake is shown in Figure 8. The figure on the left is an interference diagram generated by comparison of two dates (pre and post event) of elevation data derived using satellite mounted radar to determine the change in elevation of the ground surface elevation. The figure on the right is a contour plot of changes in elevation based on comparison of survey bench marks in the Christchurch region (pre and post event). The measurements indicate an increase in elevation of up to 450 mm on the southern, (Port Hills), side of the fault and a decrease in elevation of up to 350 mm on the northern, (Christchurch), side of the fault.

On the 13th June 2011 the Christchurch region was struck by a further significant aftershock. Another, local magnitude ML = 6.3, event occurred approximately 10 km to the east of the Christchurch aftershock (the June 2011 Christchurch aftershock epicentre is shown by a blue star on Figure 6). Again the earthquake occurred on a blind reverse fault, i.e with no surface rupture.

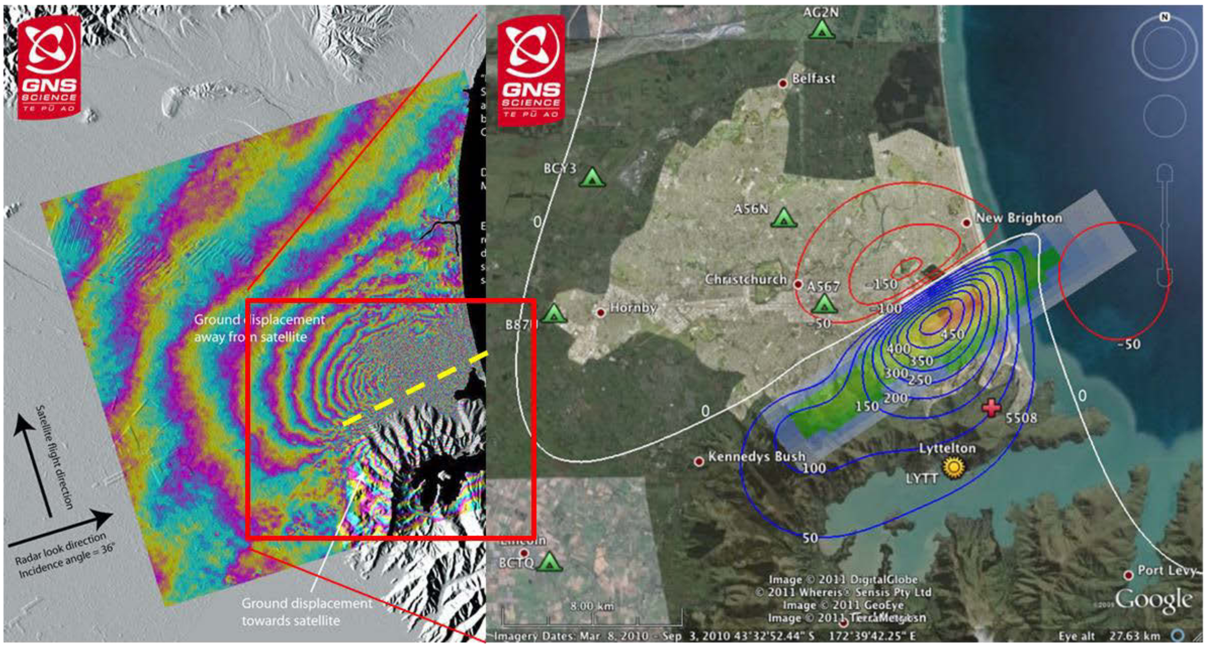


Figure 8 Distribution of permanent ground displacement associated with Christchurch earthquake (GNS, 2011)

STRONG-GROUND MOTION

The Darfield earthquake, and subsequently the aftershocks, were very well recorded by both the national-scale GeoNet network of strong-motion instruments (Petersen et al., 2010) and the Canterbury regional strong-motion network CanNet (Avery et al., 2004). Information on the instruments and the data is available via the GeoNet Web site. A useful summary of the instrumentation and data gathered following the Darfield earthquake is given in Cousins and McVerry (2010).

The measured ground motions from the Darfield earthquake were moderately high in the near field. A map of the Canterbury region showing the distribution of peak ground motions at selected strong-motion stations is shown in Figure 9. The Greendale fault rupture trace is shown as a black line on this figure. The highest peak acceleration of 1.26 g was measured at the GDLC station located immediately to the north of the fault rupture trace. Peak values of 0.88 g were measured at the TPLC station and 0.92 g the LINC station both located about 10 km east of the eastern end of the fault rupture trace. Peak ground motions within Christchurch city were more modest in the range of 0.2 to 0.3 g.

The measured ground motions from the Christchurch earthquake were generally higher in the near field than those measured during the Darfield earthquake. A map of the Christchurch region showing the distribution of peak ground motions at selected strong-motion stations is shown in Figure 10 for comparison. Note that Figure 9 and Figure 10 are at different scales. The highest peak value of 2.2 g was measured at a station located immediately to the south of the surface projection of the fault, i.e. on the hanging wall side of the fault. Peak values of 1.88 g and 1.07 g were measured within the Christchurch area and 0.92 g was measured in the Lyttleton area. Peak ground motions throughout the Christchurch area were generally high in the range of 0.5 to 0.8 g. It can generally be seen that the peak acceleration values recorded in central Christchurch were 3 to 6 or more times higher during the Christchurch earthquake than during the Darfield earthquake.

There was discussion among engineers, geologists and seismologists after the event as to why the ground motion levels generated by the Christchurch earthquake were so high. Figure 11 is a comparison between the peak horizontal acceleration values (largest horizontal component) from the Christchurch earthquake and the vertical ground motion predictive equation of McVerry et al. (2006) which was developed for New Zealand conditions. The mean and first and second standard deviations from the mean predictive equation curves are shown for the reverse fault mechanism and site class D ground conditions. It can be seen that the peak horizontal ground motions are generally in line with values that would be predicted. Figure 12 is a comparison between the peak vertical acceleration values from the Christchurch earthquake and the vertical ground motion predictive equation of Bozorgnia & Campbell (2004) which was developed for Western United States conditions, as we are unaware of a similar predictive equation for New Zealand. The mean, first, second and third standard deviations from the mean predictive equation curves are shown for the reverse fault mechanism and generic soil conditions. It can be seen that the peak vertical ground motions are generally higher in the near field than values that would be predicted but are consistent with predicted values at distances greater than about 10 km.

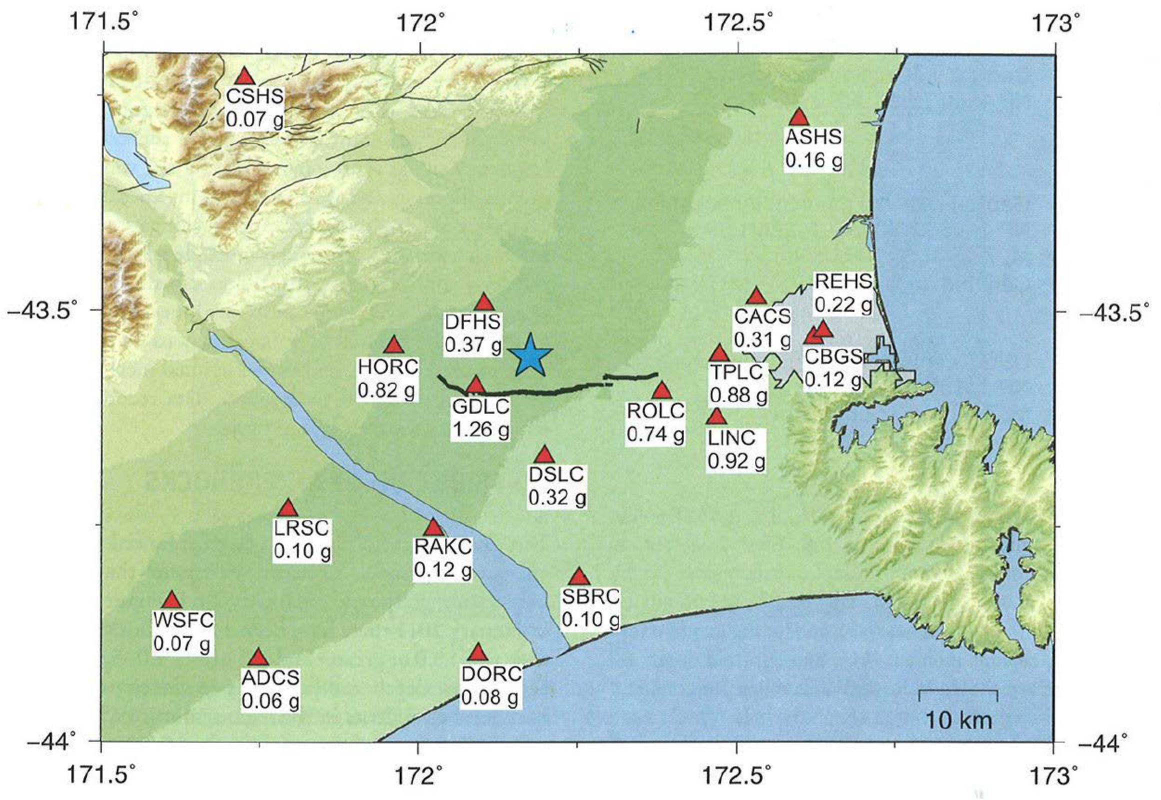


Figure 9 Map of the Canterbury region showing the distribution of peak ground motions at selected strong-motion stations (Gledhill et al., 2011)

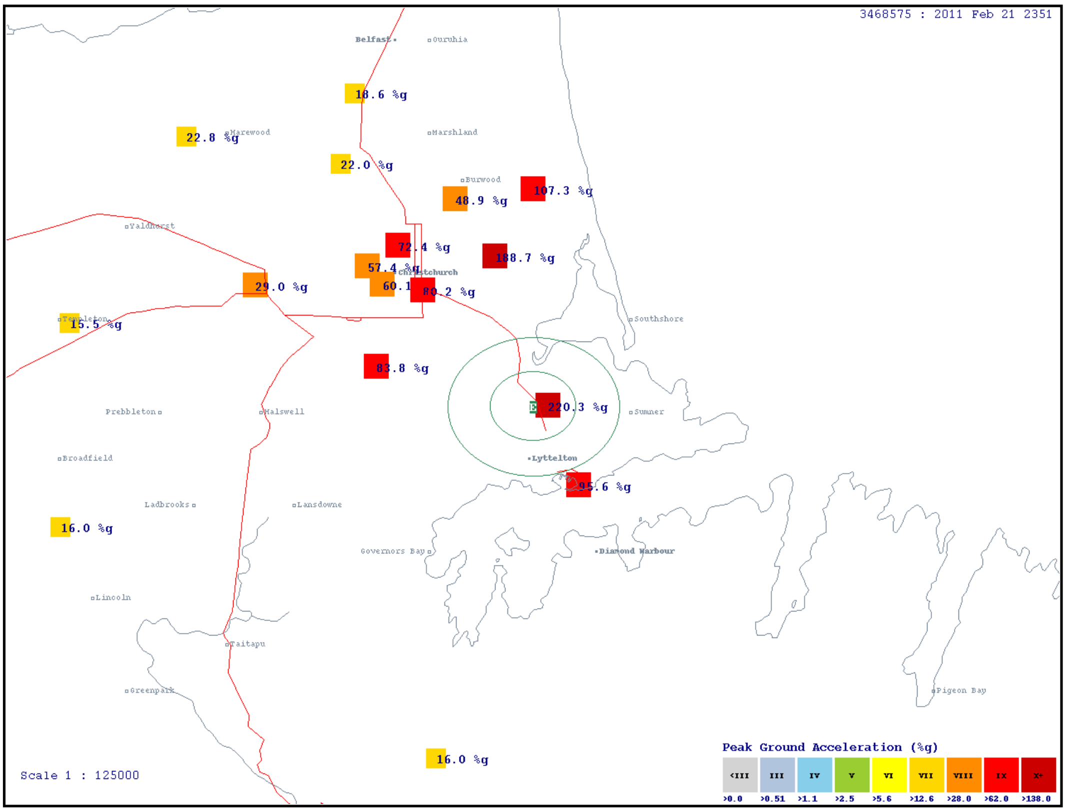


Figure 10 Map of the Christchurch region showing the distribution of peak ground motions at selected strong-motion stations (GNS, 2011)

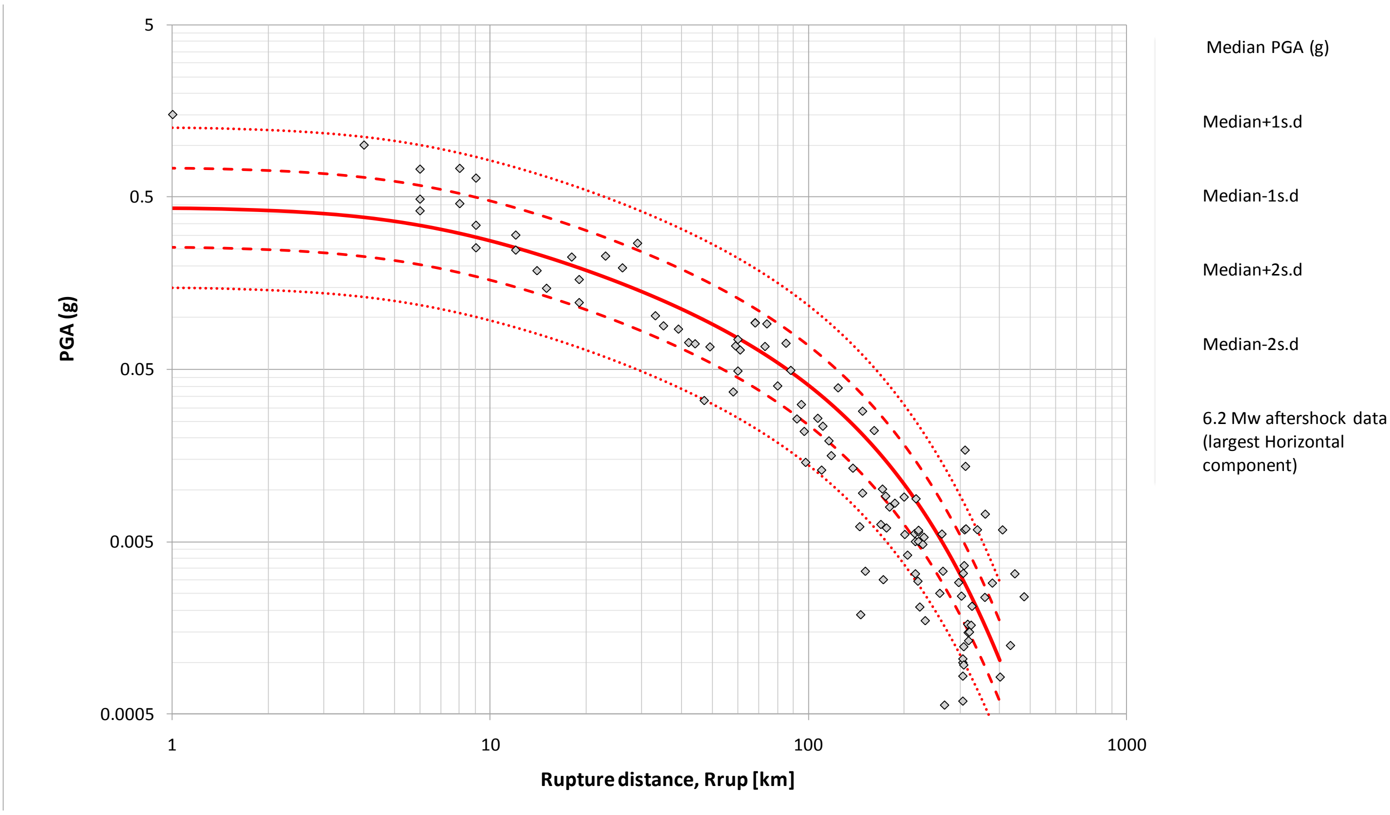


Figure 11 Comparison of peak horizontal acceleration values from the Christchurch earthquake with the ground motion predictive equation for New Zealand by McVerry et al. (2006)

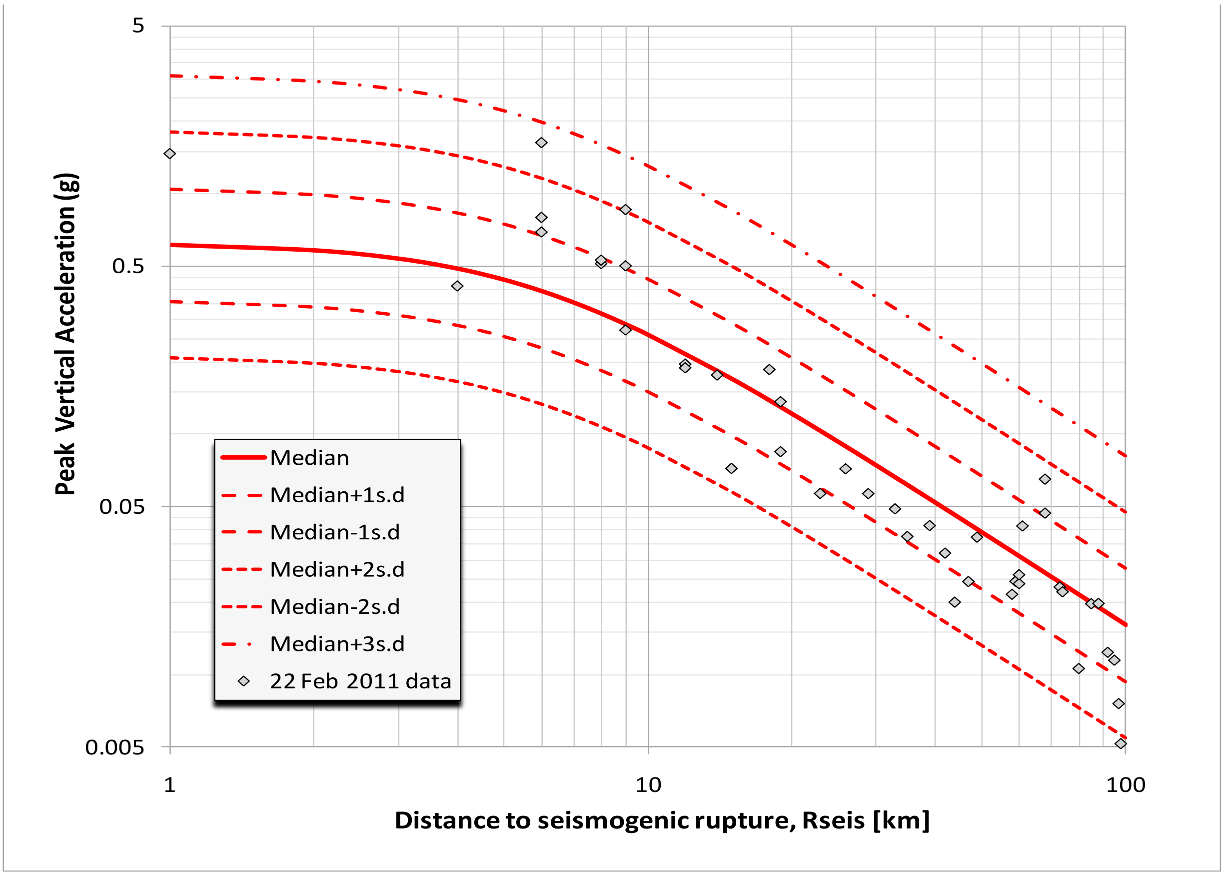


Figure 12 Comparison of peak vertical acceleration values from the Christchurch earthquake with the ground motion predictive equation by Bozorgnia & Campbell (2004)

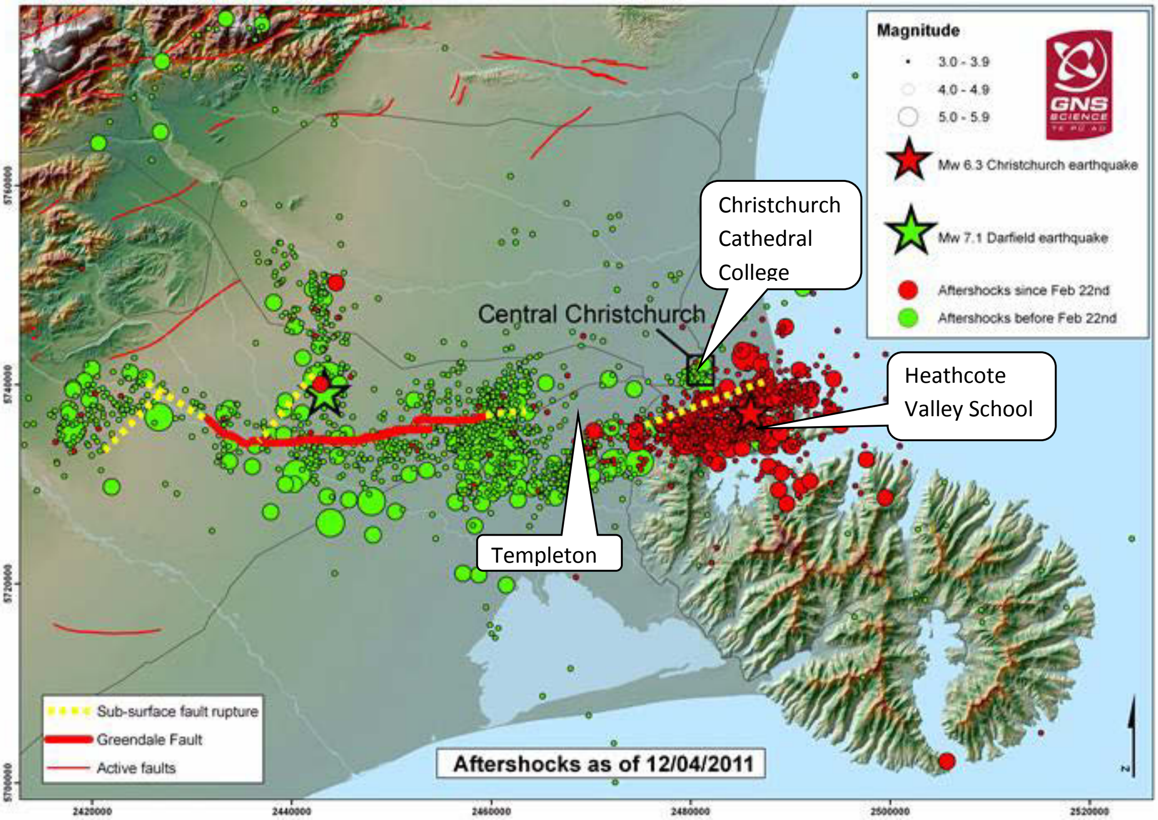


Figure 13 Map of locations of Darfield mainshock and Christchurch aftershock and aftershock sequence epicentres (GNS, 2011) also showing locations of three strong-motion instruments at Heathcote Valley School, Christchurch Cathedral College and Templeton.

Figure 13 shows the locations of three strong motions instruments a) Heathcote Valley School, which was located on the hanging wall of the Christchurch earthquake fault b) the Christchurch Cathedral College instrument, located in central Christchurch on the foot wall side of the fault, and c) an instrument located in Templeton located west of Christchurch. This last station is located approximately equally between the end of the Greendale fault (i.e. in the near field of the Darfield earthquake) and the Christchurch fault rupture (i.e. outside the near field of the Christchurch earthquake). Figure 14, Figure 15 and Figure 16 (from Bradley, 2011) show the acceleration records for both the Darfield (red lines) and Christchurch (blue lines) earthquakes recorded at these stations. In addition, the acceleration response spectra for the geometric mean of the horizontal components (solid lines) and the vertical component (dashed lines) for each earthquake record are compared with the 475 year return period design spectrum.

The Heathcote Valley School station (see Figure 14) recorded moderate to high ground motions during the Darfield earthquake, with horizontal motions about double the vertical motions (with peak horizontal about 0.6 g and peak vertical about 0.3 g). During the Christchurch earthquake the recorded motions were considerably higher (with peak vertical above 2 g and peak horizontal of about 1.3 g). The vertical motions were generally about 1.5 or more times the horizontal motions in the higher frequency range up to approximately 2 second period. This station is located on the hanging wall side of the Christchurch earthquake fault and is likely to have experienced rapid upward movement during the earthquake.

The Christchurch Cathedral College station (see Figure 15) recorded only moderate ground motions during the Darfield earthquake (with peak horizontal and vertical about 0.2 g). During the Christchurch earthquake the recorded motions were considerably higher with peak vertical motions about 0.8 g and peak horizontal motions of about 0.45 g. The vertical motions were generally about twice the horizontal motions in the higher frequency range up to approximatley 2 second period. This station is located on the foot wall side of the Christchurch earthquake fault and is likely to have experienced rapid downward movement during the earthquake.

Finally, from the Templeton station data (see Figure 16) it can be seen that the ground motions from the Darfield earthquake were larger than from the Christchurch earthquake in both the horizontal and vertical components. The vertical ground motions from the Darfield earthquake recorded at this station were particularly large. The Templeton station is located about 10 km east of the end of the Greendale Fault rupture (i.e. within the near field for Darfield earthquake). This station is also at a similar distance of about 10 km west of central Christchurch and therefore the Christchurch fault. The smaller ground-motions from the Christchurch earthquake at this location may illustrate a smaller geographic extent of the near field ground motions resulting from the smaller Christchurch earthquake compared to the Darfield earthquake.

Comparison of the response spectra for the record ground motions with the 475 year return period level spectrum indicates that the recorded ground motion hazard levels were considerably higher than 475 year hazard levels at many stations.

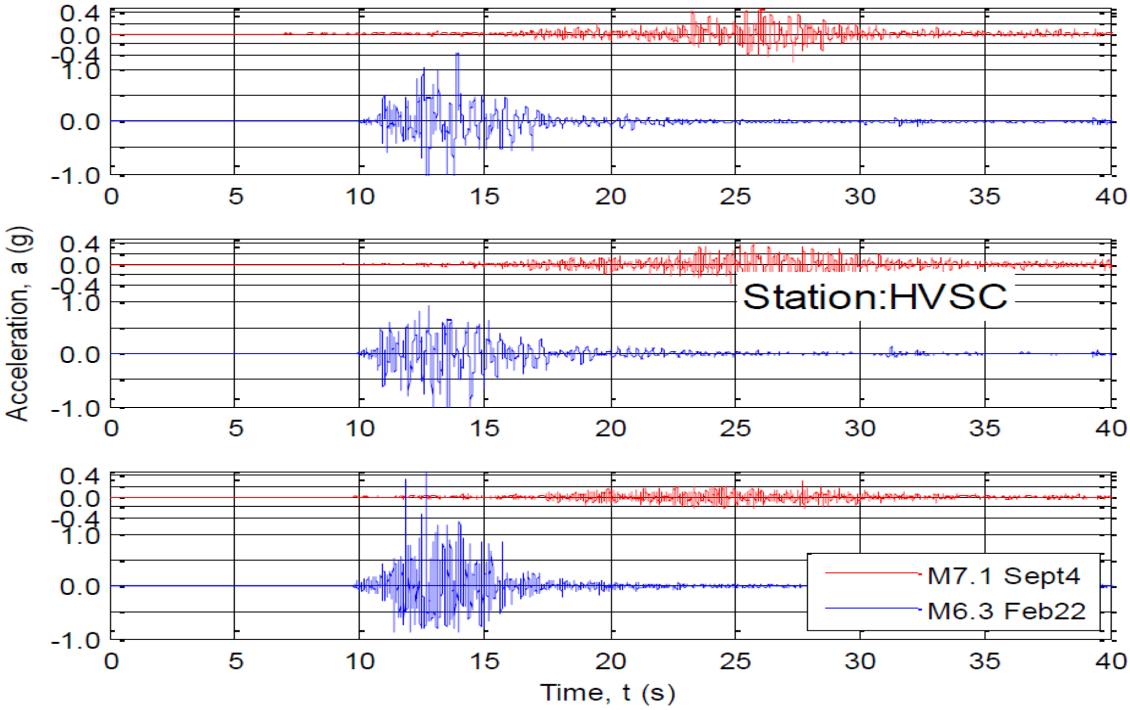
It is most likely that a combination of geological and seismological effects caused the generation of the higher than expected ground motions generated during the Christchurch earthquake:

- The fault is located very near to central Christchurch and the focus for the earthquake was shallow.
- Fault rupture occurred on a steeply dipping fault and the fault is interpreted to not have ruptured for a significant period of geological time. The rupture of healed asperities on the fault resulted in a high stress drop and higher energy release.
- Reverse faulting is generally known to cause larger ground motions particularly on the hanging wall side of the fault.
- The geometry of the fault and rupture direction may have resulted in directivity effects toward central Christchurch.
- The geology of the region comprises older and stronger basement rocks, overlain by a thick sequence of volcanic rocks beneath the Port Hills and recent sedimentary deposits beneath central Christchurch. The older basement rocks are likely to have

contributed to the high stress drop. It has been hypothesised that the thick sequence of volcanic rocks may have trapped and guided seismic waves toward central Christchurch.

- A seismic phenomenon, referred to as the "slap-down effect", interpreted to occur due to temporary separation and sudden impact closing of geological strata under seismic accelerations greater than gravity has also been put forward.

In addition, it has been suggested that some of the higher peak acceleration values may have been caused by objects falling near the recording instruments during the Christchurch earthquake. Professor John Berrill (personal communication, March 2011) reported that he had undertaken an experiment to measure the acceleration level generated by falling objects and found that peak acceleration values of up to 0.3 g may be generated by fallen objects such as desktop computers found near instruments.



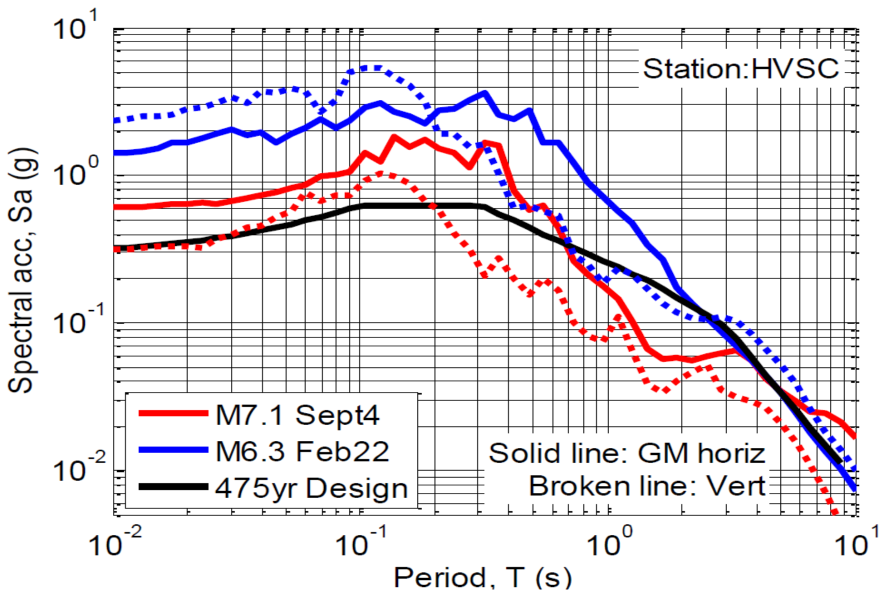
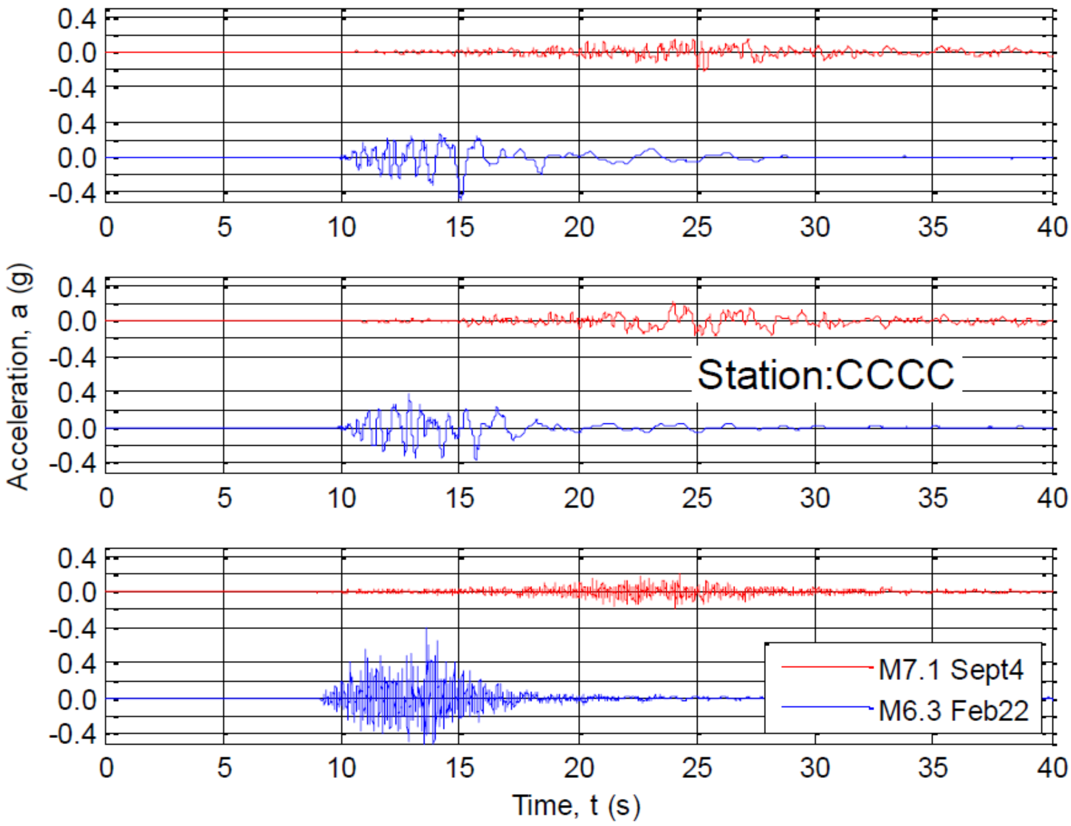


Figure 14 Heathcote Valley School (HVSC) strong motion station acceleration records and ground motion spectra (from Bradley, 2011)



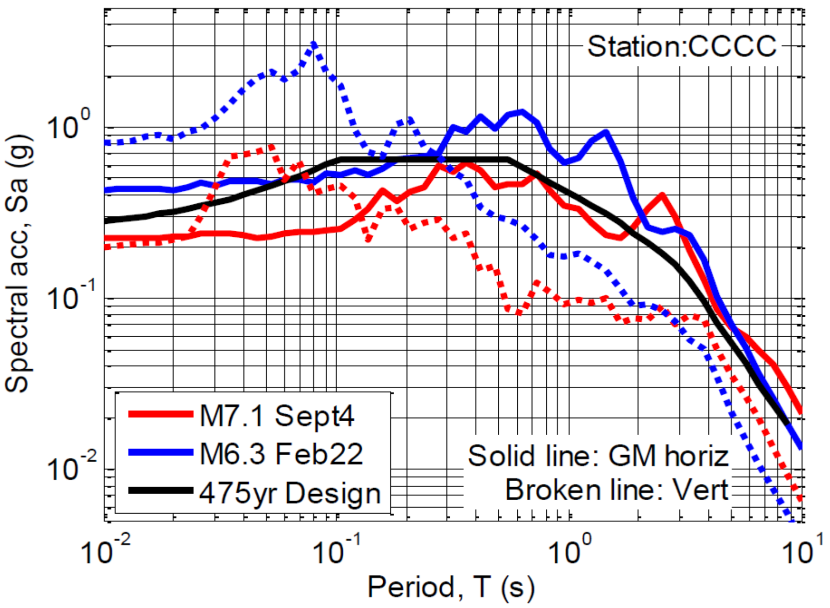
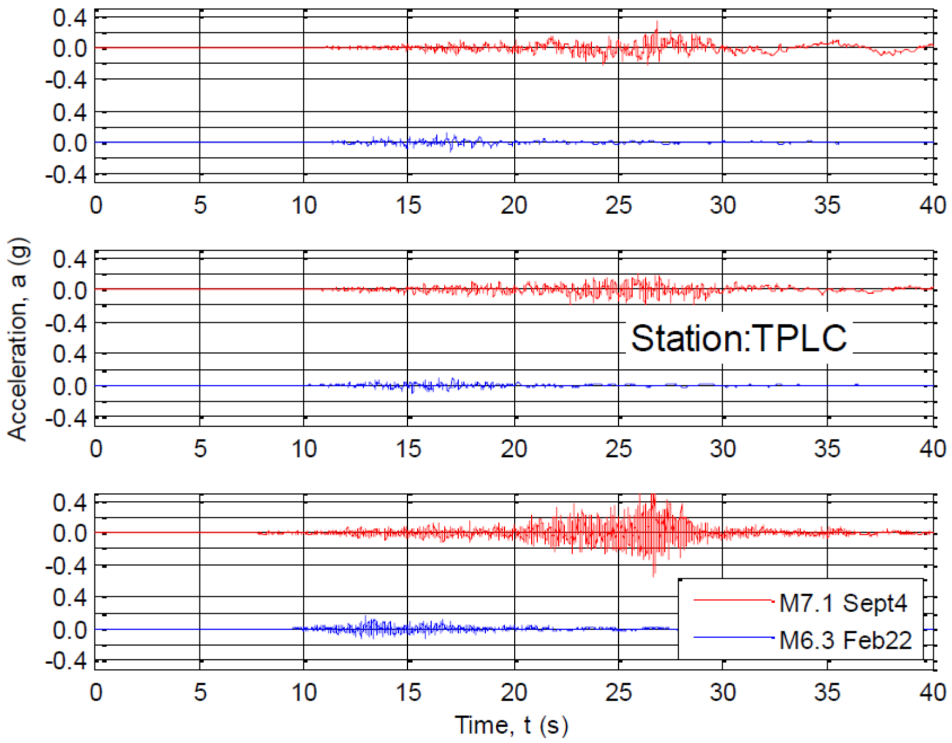


Figure 15 Christchurch Cathedral College (CCCC) strong motion station acceleration records and ground motion spectra (from Bradley, 2011)



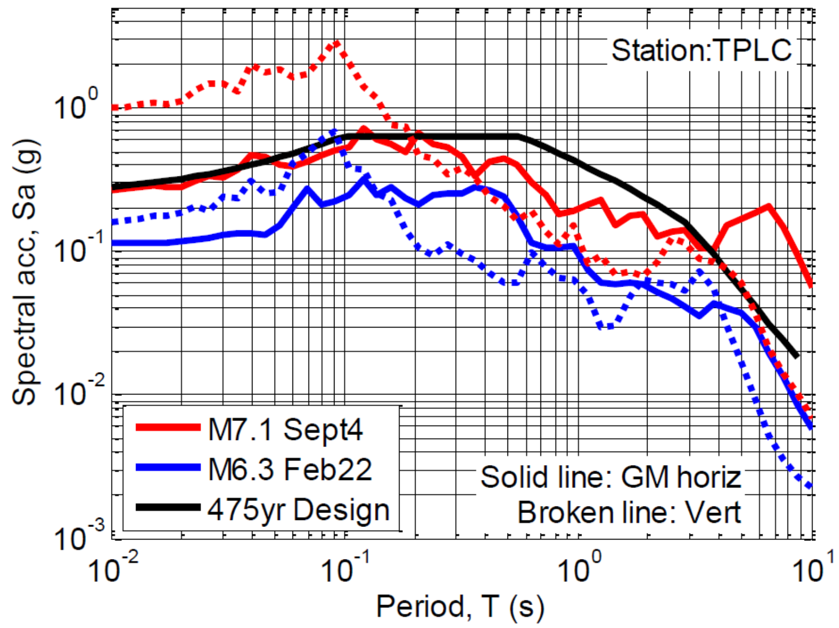


Figure 16 Templeton (TPLC) strong motion station acceleration records and ground motion spectra (from Bradley, 2011)

4. NEW ZEALAND BUILDING STOCK AND DESIGN PRACTICE

Development of Seismic Design Practice in New Zealand

Many local and international media reports following the 2010 Darfield earthquake and subsequent aftershocks have referred to the high standard of New Zealand seismic design practice. As with other areas of high seismic activity, design standards have evolved in response to earthquakes, both within the country and abroad. New Zealand earthquake engineering researchers have contributed significantly to international practice, most notably Professors Park, Paulay and Priestley, who authored and co-authored widely referenced text books on reinforced concrete design (Park and Paulay, 1975; Paulay and Priestley, 1992; Priestley et al., 1996; Priestley et al., 2007). Table 3 presents a summary of the historic development of seismic design in New Zealand, primarily based on Davenport (2004) and Beattie et al. (2008).

Current Design Practices

Current structural design in New Zealand is controlled by the Building Code (NZS1170-5), which is a section of the 1992 Building Regulations. Building Regulations are passed in accordance with the Building Act (most recent edition 2004, with amendments in 2005). The Building Code is a performance-based set of requirements that dictate the high-level performance objectives with which new and existing buildings must conform over their life.

New Zealand Standards form acceptable reference documents to demonstrate Building Code compliance. The most relevant New Zealand standards for the design of new buildings are listed in Table 4.

Table 5, excerpted from a larger table in (PCFOG, 2009), summarises the expected performance of normal importance (importance level 2) buildings under different levels of earthquake ground motion, given in terms of the annual probability of exceedance (the reciprocal of the return period). It does not have official status, but is considered to be consistent with the performance requirements of the Building Code (PCFOG, 2009). Levels of ground motion to use for design according to NZS 1170.5 are calculated based on the design life and importance level of the structure. For normal importance structures with design life of 50 years, ultimate limit state (ULS) calculations are carried out for a 1/500 (= 0.2%) annual probability of exceedance, and serviceability (SLS) checks are carried out for a 1/25 (= 4%) annual probability of exceedance.

Table 3 – Summary of development of NZ seismic design practice (after Davenport (2004) and Beattie et al. (2008))

1848	Marlborough earthquake, M7.5
1855	Wairarapa earthquake, M8.2
1931	February, Hawke's Bay earthquake, M7.8. This earthquake destroyed much of the city of Napier and caused 256 fatalities (which remains the highest from any NZ natural disaster). Committee convened to prepare recommendations for seismic-resistant construction; Draft General Earthquake By-Law published in June, which gave design loads (uniform up height of building), and requirements for checking and supervision of design and construction.
1935	New Zealand Standard Model Building By-Law published. Local bodies could choose to adopt or not; designers in major cities (and Napier) made use of the standard. Suggested that new public buildings should have reinforced concrete (RC) or steel frames.
1955	Revision of NZ Standard Model Building By-Law. Adopted a linear load profile up height of building.
1965	Further revision of NZ Standard Model Building By-Law. Introduced seismic zonation into three zones, and included dynamic response analysis (response spectrum) as an option for special structures. Introduced concept of ductility (for RC structures, structural ductility 4, viscous damping 10%), but did not suggest detailing practice to allow this ductility to be achieved.
1968	New Zealand Society for Earthquake Engineering (NZSEE) convenes its first meeting.
1969	Concept of "Capacity Design" first presented by John Hollings in two papers of the Bulletin of the NZSEE. Professors Park and Paulay continued to develop the conceptual and experimental background to capacity design at the University of Canterbury in the late 1960s through the 1970s and beyond, including the publication of their book, *Reinforced Concrete Structures*, published in 1975 (Park and Paulay, 1975).
1976	Code of Practice for General Structural Design and Design Loadings for Buildings published, including capacity design requirements. Material standards lag loadings standard: 1977 interim steel structures standard incorporating ductility and capacity design requirements published; 1982 concrete standard published.
1978	Code of Practice for Light Timber Framed Buildings Not Requiring Specific Design published. Establishes qualitative rules to aim for habitability of houses following the design earthquake.
1981	William Clayton Building completed in Wellington, New Zealand's first base isolated building. Uses the lead-rubber bearing device invented by New Zealander, Bill Robinson.
1992	Revision of Code of Practice for General Structural Design and Design Loadings for Buildings published. Code ultimate limit state design spectrum based on 10%-in-50 year uniform hazard spectrum. Includes serviceability requirements for seismic design.
2004	Joint Australian/New Zealand loadings standard published. Section 5 (applicable to New Zealand only) updates the seismic load provisions from the 1992 standard. Includes amplification factor for buildings within 20 km of a major fault to account for near-field effects. Informative appendices give more information about implementation of full capacity design requirements, including discussion of unidirectional plastic hinges (for gravity-dominated beams). Concrete materials standard updated in 2006.
2004	New Building Act published. Requires territorial authorities to adopt a policy on earthquake prone buildings by 31/5/2006 and revisit every 5 years. Applies to all buildings except residential buildings.

Table 4 – Relevant New Zealand standards for design of new buildings

Loadings; Seismic Actions	AS/NZS 1170:2002; NZS 1170.5:2004
Concrete buildings	NZS 3101:2006
Steel buildings	NZS 3404:1997 (currently under revision)
Timber buildings	NZS 3603:1993
Timber-framed buildings (up to 3 storeys)	NZS 3604:1999 (2011 version just published)
Non-engineered concrete masonry	NZS 4229:1999
Engineered concrete masonry	NZS 4230:2004

Table 5 – Seismic performance expectations for buildings in New Zealand (PCFOG, 2009)

Earthquake event definition (indicative annual probability)	Tolerable impact level	Description of impact
>1/2500	Extreme	Building collapse
1/1000	Very severe	Building unsafe to occupy for one year or more. Major and extensive damage to structure and building fabric. Not repairable. Contents not salvageable. Access denied for an indefinite period. Building function ceases.
1/500	Severe	Building unsafe to occupy for up to one year. Major damage to structure and building fabric, but capable of repair. Most contents seriously affected. Building function extensively affected. Unassisted evacuation possible.
1/100	High	Building function affected for up to seven days. Moderate but repairable damage to structure. Damage to building fabric requires replacement of some items. Most contents affected. Access inhibited. Most buildings safe to occupy after clearance by authorities.
1/25	Mild	Building function maintained. Little or no damage to structure. Minor damage to building fabric. Some contents affected. Building fully accessible and safe to occupy.
Every day	Insignificant	No significant effects on building elements, occupants or functions

NZS 1170.5 maps the distribution of seismic hazard in the country in terms of a zone factor, *Z*, corresponding to the expected zero-period spectral acceleration, normalised by gravity, for a 1/500 annual probability ground motion on hard rock or rock. The values of *Z* range from 0.13 to 0.60 for towns and cities in New Zealand; the value for Christchurch is 0.22. The distance to major faults is also tabulated, to determine a period-dependent near-fault factor that amplifies the elastic site spectrum for periods greater than 1.5 seconds and fault distances less than 20 km. The nearest major fault considered for Christchurch is the Alpine fault, around 100 km away, and therefore the near-fault factor is not applied to design in Christchurch. The elastic site spectra for the 1/500 annual probability ground motion of Christchurch is shown in Figure 17 for different site conditions.

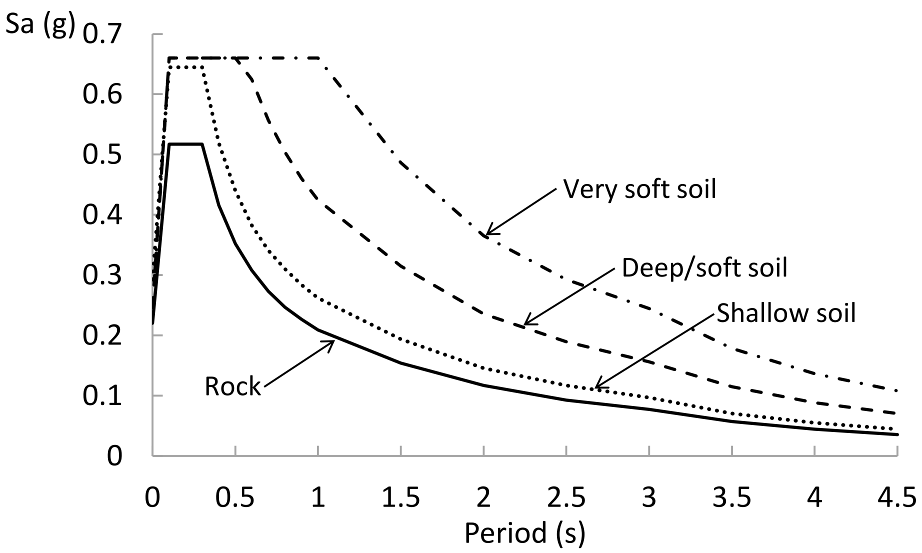


Figure 17 – Elastic site spectra for Christchurch (NZS 1170.5)

For ultimate limit state calculations, the design spectrum is modified by two factors: k_μ and S_p. The former is an explicit function of the structural ductility, μ, and the latter is a structural performance factor which recognises the satisfactory performance of modern buildings in previous earthquakes. The equal displacement rule is applied for periods greater than 0.7 seconds (for all but very soft soil sites), such that $k_\mu = \mu$, and for shorter periods a transitional relationship is given, such that $k_\mu = 1$ at zero period. The structural performance factor is generally taken as 0.7. Therefore, when considering NZS 1170.5 with Eurocode 8's *q* factor and the *R* factor used in US practice, the ratio of k_μ/S_p appropriate for comparison, is generally $\mu/0.7$ for long periods and typical structures. The maximum allowable ductility values, μ, and $\mu/0.7$ for various construction materials and structural systems are shown in Table 6. Note that material ductility limits are also given in materials standards, and may govern over the allowable values given in Table 6.

Table 6 – Maximum allowable ductility factors to New Zealand Standards (NZS 1170.5)

Material and Structural System	**μ**	**μ/0.7**
RC limited ductility moment frames	3.0	4.3
RC ductile moment frames	6.0 (5.0 for precast)	8.6 (7.1)
RC limited ductility structural walls	3.0	4.3
RC ductile structural walls	2.5→6.0*	3.6→8.6*
Steel limited ductility moment frames, CBFs	3.0	4.3
Steel ductile moment frames, CBFs, EBFs	6.0	8.6
Limited ductility RM	2.0	2.9
Ductile RM	4.0	5.7
Ductile timber	4.0	5.7

*depends on aspect ratio of wall(s)

The various standards noted in Table 4 contain provisions to ensure that design levels of displacement ductility can be reliably obtained prior to structural collapse, and that the performance objectives outlined in Table 5 can be satisfied. In particular, capacity design procedures are implemented explicitly and transparently – arguably more so than in European and US codes. Notably, NZS 3101:2006 contains a normative appendix on the design of ductile jointed precast concrete structural systems, based on the US PRESSS programme (Priestley et al., 1999), and subsequent work by (amongst others) Dr Stefano Pampanin's research team at the University of Canterbury. This demonstrates a benefit of the relatively small New Zealand engineering community – state-of-the-art research can be relatively quickly codified and disseminated without significant backlash or code inertia that can be an obstacle to best practice in other countries. This is also reflected in the strong desire of local engineers and academics, expressed at public meeting at the time of the EEFIT team's visit, to learn lessons from the Christchurch earthquake, and to incorporate the most up to date design practice (both local and international) into the rebuild of the city.

Policy for Strengthening of Existing Buildings

The 2004 Building Act obliged territorial authorities (TAs) to develop a policy regarding "earthquake prone buildings" (EPBs) by the end of May, 2006. For the purpose of this legislation, an EPB is defined as a building with less than 33% of the capacity of one designed according to the current standards, and that would be likely to collapse causing injury or death, or damage to other property. Residential buildings of one-storey, or containing fewer than three household units, are excluded. TAs must reassess this policy every five years. The New Zealand Society for Earthquake Engineering developed a guideline document (NZSEE, 2006) that may be used by TAs in the development of their policy.

Christchurch City Council adopted an "Earthquake-prone, dangerous and insanitary buildings policy" in 2006. An updated version from 2010 (CCC, 2010), released following the Darfield earthquake, introduced timeframes within which strengthening works must be carried out, of 15–30 years, based on the importance level defined in AS/NZS 1170.0. The lower limit (15 years) applies to buildings with special post-disaster functions, and the upper limit (30 years) applies to “normal importance” buildings. Although, such timescales seem long, but they were set in accordance with the NZSEE (2006) recommendations, and against the backdrop of an estimated NZ$169 million bill to strengthen all EPBs to the minimum 33% of current code level (the cost estimate was made before the 2010 Darfield earthquake). Target levels for strengthening would be developed on a building by building basis, but would generally follow the 67% of current code recommendation of NZSEE (2006) (i.e. a building with strength < 33% of new building standard must be strengthened to > 67%; a building with strength > 33%, but possibly < 67%, need not be strengthened).

Structural typology and associated usage

Table 7 presents a summary of different construction typologies and materials used in New Zealand, predominantly adapted from Cousins (2009) and other references noted in the table.

Table 7 – Construction materials, typologies and uses in New Zealand

Typology	Description	Primary Use(s)
Unreinforced Masonry (URM)	Walls constructed of solid clay bricks in cement mortar. Roof and floor framing in timber. Most are 1- and 2-storey buildings. Almost all NZ URM buildings are pre-1940 construction, due to widespread damage in the 1931 Napier earthquake, and the effects of the Great Depression and World War 2 on the construction industry. Legislation implicitly discouraged URM construction as early as 1935, imposed further restrictions in 1965 and explicitly forbade it in 1976 (Russell and Ingham, 2010).	Heritage (pre-1940s) commercial, residential
Reinforced Masonry (RM)	Walls constructed of hollow concrete masonry blocks in cement mortar, with reinforcement and grout in cells. RC slabs and timber roof framing.	Commercial, residential, industrial
Reinforced Concrete (RC) Moment Frames	RC columns, beams, floor and roof slabs (cast in place or precast). Changes in seismic design practice, especially in the 1976 loadings code and subsequent concrete standard, mean that this category is further subdivided into pre-1976 and post-1976 (although further subdivisions would be possible due to continuing evolution of design and construction practice; see text). Park (2002) provides an excellent summary of the use of precast concrete in New Zealand (for moment frames, structural walls and, particularly, floor systems).	Commercial, industrial
RC Structural Walls	RC walls, floor and roof slabs, and RC columns to carry portion of gravity load. Cast in place or precast. Low-rise buildings may be constructed with timber roof framing (no RC roof diaphragm). 1–3-storey precast walls may be constructed using tilt-up wall panels, anchored at foundations and interconnected; typically RC floors and steel or timber truss or portal frame roof. Note we adopt the usage “structural wall” rather than “shear wall”, based on the recommendations of Paulay and Priestley (1992), to avoid a connotation of brittle shear-dominated behaviour.	Commercial mid-to-high-rise buildings, and commercial, residential and industrial low-rise buildings
Steel Moment Frames / Concentrically Braced Frames (CBFs) / Eccentrically Braced Frames (EBFs)	Steel columns and beams. Floors and roofs in metal deck with concrete topping or cast-in-place concrete slabs. Single storey industrial buildings may be lightweight steel portal frames in short direction and diagonal steel tie-rods in longitudinal direction. RC was preferred to steel for a number of years by the NZ construction industry, following labour disputes during the construction of the BNZ Centre in Wellington in the 1970s and 1980s. Steel construction is returning to favour, and most steel buildings in Christchurch are of recent construction and therefore designed according to recent seismic standards (Bruneau et al., 2011).	Commercial, industrial. Lightweight framing used predominantly in industrial warehouses and some residential
Timber frame buildings	Timber stud frame construction with plywood sheething. Walls clad in timber weatherboards, brick veneer, stucco or corrugated iron. Timber roof framing with corrugated iron, clay tile, concrete tile or sheet metal tile cladding. Thurston and Beattie (2008, 2009) have recently carried out experimental testing on 1- and 2-storey timber framed brick veneer buildings, and demonstrated satisfactory performance. However, these tests made use of modern anchor details which may not be typical of the majority of brick veneer houses in New Zealand.	Residential

5. PERFORMANCE OF BUILDINGS

TIMBER BUILDINGS

Much of the residential building stock in Christchurch comprises one and two storey timber structures. These buildings are primarily timber frame and timber shear wall construction, with non-structural walls of timber weatherboards and tiles or corrugated metal roofing. Foundations observed by the team were most commonly concrete slab, with the use of wooden pile foundations in a limited number of older structures. It is common for timber residential buildings to have an external masonry chimney breast and stack.

Overall, the timber building stock was structurally resistant to ground shaking in this event, despite the high ground accelerations recorded. Examples of damage to timber residential buildings are shown in Figure 18 and Figure 19.

The damage observed by EEFIT was primarily driven by ground failure around or beneath the structure; namely the differential movement of separate structures and fracture of concrete block foundations due to lateral spreading and settling of foundations. Foundations are discussed further in the geotechnical section, later in the report.

Other commonly observed damage included damage to masonry components of the building: falling chimney stacks, damage or collapse of masonry facades or to external chimney breasts; and roof tile damage. Several instances of soft storey failure in a timber building were also observed.

EEFIT carried out a systematic survey of residential buildings in the proximity of accelerometers. The objective of such a survey was to determine accurate building vulnerability data coupled with good estimates of earthquake ground shaking. Often in field studies, a large amount of effort is invested in collecting damage data, but, without an actual recording of ground shaking, the uncertainty in the intensity of the ground motion is large. More information about the residential damage survey is presented in Appendix A, including a description of the methodology, results and known limitations of the study.

The observations of residential building damage made by EEFIT support observations reported by large-scale residential-specific surveys made across Christchurch (Beattie, 2011).

Figure 18 – Masonry wall and chimney collapse on residential timber building. This house has a double-skin masonry wall where the chimney was situated. External assessment shows the timber frame of this building to have performed well, although the masonry wall and chimney have partially collapsed (possibly causing roof damage).

(a) (b) (c)

Figure 19 – (a) Collapse of a stone chimney breast on a timber frame house; (b) lateral spreading towards the river (left of photo) has resulted in differential movement and separation at the connection between the small extension and the main house; (c) soft storey failure at the ground floor level of a two storey timber frame house, as well as collapse of masonry facade.

Reinforced Concrete Buildings

Reinforced concrete (RC) is the most widespread construction material used in Christchurch, at least in the Central Business District (CBD) and other commercial areas. The majority of reinforced concrete buildings performed reasonably, in that they satisfied the performance objectives outlines in Table 5. A few notable failures were reported by local and international news media, especially the Canterbury Television (CTV) and Pyne Gould Corporation (PGC) buildings, which were responsible for at least 95 and 15 of the total fatalities, respectively (The Press, 2011b). Most failures could be attributed to irregularity in plan and/or elevation, or inadequate detailing (moment frame beam-column joint reinforcement; wall boundary zones). Researchers at the University of Canterbury had been investigating retrofit solutions for Christchurch's earthquake prone reinforced concrete building stock prior to the event, and therefore they have been well prepared for studying and disseminating information about building performance (Kam, 2011; Kam *et al.*, 2011).

Several modern RC moment frame buildings were visited by the team – mainly from the outside, but in some cases they were able to participate in Level 2 (internal) assessments. For the most part, moment frame buildings performed well, and most damage was caused by differential settlement (see Foundations Performance section). Minor flexural cracking was observed in beams (Figure 20) and at column bases (Figure 21a), although in some cases flexural hinging was observed at the top of ground storey columns, along with shear cracks up the height of columns, in multi-storey buildings (Figure 21b and c). This minor cracking would not be expected to lead to a significant reduction in the capacity of the building to withstand subsequent earthquakes.

(a) (b)

Figure 20 – Beam damage observed on Level 2 (internal) inspection of building on Victoria Street. (a) Internal beam-column joint damage exposing transverse joint reinforcement; (b) flexural cracks on underside of beams at corner beam-column joint.

(a) (b) (c)

Figure 21 – Column damage observed on Level 2 (internal) inspection of building on Kilmore Street. (a) Flexural crack at ground floor column base; (b) flexural cracking at top of column; (c) potential shear cracking up height of column (note: cracks drawn on column in b and c).

Poor performance of some RC buildings in the earthquake was due to irregularity – either horizontal or vertical – of the lateral force resisting system. A typical example of horizontal irregularity is the building shown in Figure 22(a). The lateral system of this building comprised structural walls on the two sides, moment frame on the street frontage, and infilled moment frame on the back wall. This would have shifted the centre of stiffness towards the rear of the building, and the subsequent torsional demand led to heavy damage to the columns and beam-column joints of the front moment frame (Figure 22b). This building has subsequently been demolished. The CTV and PGC building collapses, mentioned earlier, also appear to have been at least partially due to horizontal irregularity.

(a) (b) (c)

Figure 22 – Problems with structural irregularity. (a) and (b) TVNZ building with open street frontage and infilled frame at rear. (c) Grand Chancellor Hotel building, showing overhang of driveway at ground floor level.

There was much anecdotal evidence from local engineers about poor structural performance due to vertical irregularity, including a prominent hotel building in the CBD with severe damage to a transfer structure at the 1st floor level. The most notable example of vertical irregularity leading to inadequate seismic performance was the 26-storey Grand Chancellor Hotel building – one of the tallest buildings in the Christchurch CBD (Figure 22c). On the day following the earthquake, rescue teams were evacuated from the area around the building, as it was reported to have leaned 1 metre towards the east over a 10-minute period (New Zealand Herald, 2011a). The building was subsequently stabilised for eventual demolition. The Earthquake Engineering Research Institute (EERI) reconnaissance team reported that the building lean was due to the failure of a transfer structure at the 7th floor, which was required to allow an overhang over a driveway at the ground level. The driveway is visible at the bottom right of Figure 22(c).

Various degrees of damage were observed to RC structural walls, from no visible cracking, to moderate cracking in plastic hinge zones (Figure 23), to significant crushing of concrete and buckling of longitudinal bars in boundary zones (Figure 24 and Figure 25). It is interesting to contrast the latter two figures, which show similar boundary zone damage, but different quantities of transverse reinforcement.

(a) (b)

Figure 23 – Moderate cracking in RC structural walls. (a) Cracks drawn on internal core wall in 7-storey building on Kilmore Street; (b) cracking at the base of a wall on Victoria Street. The building in (b) is an “indicator building”, which means that following a significant aftershock, this would be one of the first to be inspected to assess whether more widespread damage is likely.

(a) (b)

Figure 24 – Significant damage to a wall with well-confined boundary zones on Victoria Street. (a) Wall with core concrete removed for repair operation, and (b) close-up of boundary zone. EERI reconnaissance team concluded that this was a wall buckling failure rather than a flexural boundary zone failure due to large compression strains.

(a) (b) (c)

Figure 25 – Significant damage to a wall with poorly-confined boundary zones on Oxford Terrace. (a) Whole wall; building had settled 300–400 mm towards the right of picture; (b) close-up of boundary zone; (c) propping (behind wall) was in place due to damage in September Darfield event.

The wall shown in Figure 24 was from a 7-storey building, of apparently modern construction, with moderately reinforced but well-confined boundary zones. Note that damaged concrete had been removed from the boundary zone reinforcing cage at the time of the visit, and repair works had been designed by local structural engineers. Interestingly, the corresponding wall on the other side of the apparently-symmetric building was not heavily damaged. This is presumably due to openings in the floor slabs around this other wall which would have restricted the diaphragm force that could be transferred and therefore overloaded the damaged wall. The EERI reconnaissance team concluded that this could have been a wall stability failure (i.e. buckling of the whole wall) rather than a failure due to insufficient boundary zone transverse reinforcement, which could have been a result of the large vertical ground motions observed.

The wall shown in Figure 25 was from another 7-storey (approx.) building of 1980s construction. The boundary zone in this wall contains ties around only the last two longitudinal bars at a much wider spacing than would be permitted by modern design standards. Note that the undamaged section to the left of the damaged area is a non-structural blockwork wall. The propping visible in the photo had been in place since the September Darfield earthquake (Kam Y. Wen, 2011; personal communication). The building settled around 300–400 mm towards the right-hand corner of the picture, due to a combination of damage to the walls and differential settlement.

A large proportion of the reinforced (and prestressed) concrete used in New Zealand is precast, especially precast hollowcore floor systems, which were first produced in New Zealand in the late 1960s (Park, 2002; PCFOG, 2009), and precast stair units. Over the last decade, research has been carried out to investigate the adequacy of existing seating details (PCFOG, 2009). The most notable staircase failure in the earthquake was over several stories of the 17-storey Forysth Barr building. Local media reported (New Zealand Herald, 2011b) that this failure was due to fixing of the stair units at both the top and bottom, rather than the typical detail which allows movement at the bottom so as not to form a strut between levels. The collapsed stairwell prevented evacuation of the occupants, and impeded search and rescue efforts, clearly not satisfying the “Unassisted evacuation possible” objective from Table 5. Figure 26(a) shows tenants being assisted entry to the building by crane so they could retrieve possessions (note the broken windows that have been labelled at each level by search and rescue teams).

(a) (b)

Figure 26 – (a) Forsyth Barr building where stairwells collapsed, impeding evacuation and search and rescue efforts. (b) Staircase detail exposed by damage on building on Kilmore Street; angle section provides 60 mm seating, and around 15 mm permanent offset (some of which may be construction tolerance) measured after earthquake.

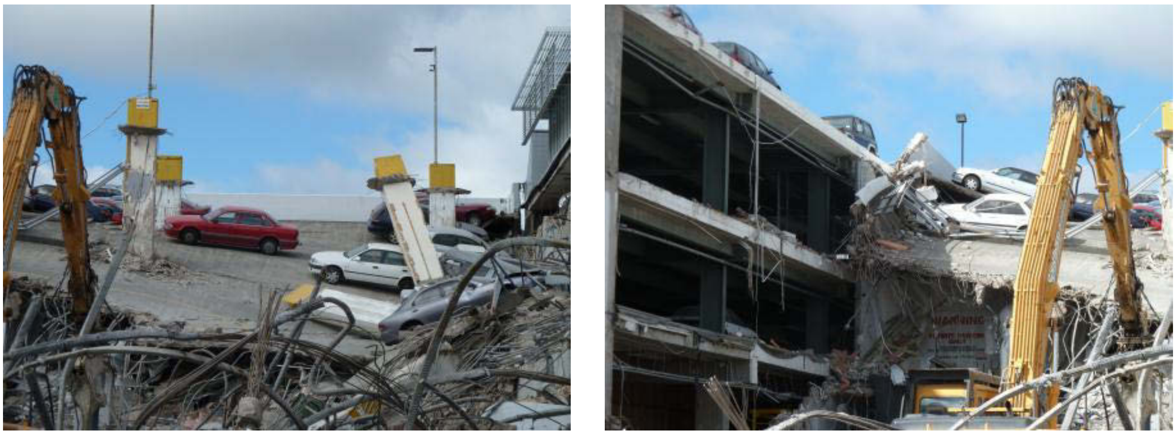
The team also observed damage at the top of stairwells as part of the Level 2 (internal) inspections. Figure 26(b) shows the exposed seating detail, with 60 mm of seating, but 15 mm of permanent offset (some of this could also be construction tolerance). The peak movement during the earthquake would of course have been larger than this.

Precast concrete is also used in New Zealand for beams, columns and structural walls (Park, 2002). Figure 27(a) shows a parking building with precast post-tensioned concrete beams (the anchorages are evident on the face of the column). This building appeared to have sustained very little damage in the earthquake. Larger precast structures, including the AMI stadium shown in Figure 27(b), also seemed to have performed well with very little external damage, although it has been reported that internal damage is more significant (The Press, 2011a).

(a) (b)

Figure 27 – Precast concrete structures. (a) Parking building on Gloucester Street with post-tensioned, precast beams, showing little damage. (b) AMI Stadium, with little damage visible from the outside (despite significant liquefaction), but reportedly damaged internally (The Press, 2011a).

In contrast to Europe, flat slab construction is rare in Christchurch, although a few examples were noted. The few flat slab structures encountered had variable performance, ranging from damage confined to plastic hinges in columns to catastrophic collapse. A notable punching shear failure occurred in the car parking building on Eaton Place, shown in Figure 28(a). This structure also comprised a 3-storey structure supported over the street level. The top two levels of this section collapsed onto the lower level, which did not collapse – it had been demolished and removed at the time of the EEFIT team visit. The adjoining steel moment frame building, shown in Figure 28(b), did not appear to have suffered significant damage.



(a) (b)

Figure 28 – Eaton Place parking building. (a) Punching shear failure in RC building; (b) no apparent damage to adjacent steel moment frame building.

Steel Buildings

As noted in Table 7, there are relatively few steel buildings in Christchurch (and New Zealand generally). This is partly due to labour disputes in the 1970s and 1980s, and a general favouring of reinforced concrete by the New Zealand construction industry. Only two examples are discussed in this section; more detailed observations have been released in draft form by Bruneau et al. (2011). They report on a number of steel structures, including a few interesting examples of poor steel performance: most notably an eccentrically braced frame (EBF) link failure (that they describe as the first such example in the world), and the failure of a bracing connection in a concentrically braced frame (CBF), both in parking buildings. Many of the authors of these draft observations were also authors of a paper on the performance of steel structures in the Darfield earthquake, published in the Bulletin of the New Zealand Society for Earthquake Engineering (Bruneau et al., 2010).

The 22-storey Pacific Tower is the tallest building in Christchurch – at 86 metres tall it is 1 metre taller than the Hotel Grand Chancellor if its 13 metre communication mast is included (Bruneau et al., 2011). It was completed in 2010 (Figure 29a), and was under construction when the Google Streetview photos were taken (Figure 29b). The lateral system comprises perimeter EBFs on the lower six levels, transferring to EBFs around a central core above this. Notably, the tower design was governed by drift limits, resulting in an effective design ductility of 1.5. It is therefore not surprising that damage to the building was slight – no damage was visible from the outside, although Bruneau et al. (2011) report evidence of yielding and residual deformation of EBF link beams (see Figure 29c). Local lightweight bracing around an automated parking elevator was also reported to have failed.

(a) (b) (c)

Figure 29 –22-storey steel EBF/Moment Frame building, Pacific Tower, 2010; (a) EEFIT photo from visit; (b) construction photo from Google Streetview, showing perimeter EBFs in lower levels; (c) evidence of EBF link paint flaking due to yielding, and residual deformation, partially obscured by services (Michel Bruneau, from Bruneau et al., 2011).

As part of detailed Level 2 inspections, the team also visited a 7-storey building in the CBD with a mixed RC moment frame (ground level) and steel moment frame (higher levels) lateral system (Figure 30a). The building suffered differential settlement due to liquefaction, and evidenced by distress in ground floor columns and ground-bearing slab. There was no evidence of any damage to the steel moment frames at higher levels, including on the floor that had fully exposed beams and columns, due to fit-out (albeit covered in fireproofing; see Figure 30b). An area of fireproofing was removed (see Figure 30c), which showed no signs of buckling in beam flanges; however, a more detailed inspection would have required fireproofing removal from the beam web, beam-column joint, and columns.

(a) (b) (c)

Figure 30 – (a) 7-storey building in Christchurch CBD with concrete moment frame system at ground floor and steel moment frame at higher levels. (b) First floor was being fitted out so beams and columns were exposed, albeit covered in fireproofing. (c) Removal of fireproofing at the end of a beam showed no signs of flange buckling or connection damage.

Unreinforced Masonry (URM) Buildings

The survey of unreinforced masonry (URM) structures mainly focused on the areas of the CBD and Lyttelton, where this constructive typology constitutes a large percentage of the building stock. Masonry structures were also observed in various residential areas, although in smaller number since timber frame has far wider application for residential construction. Surveyed buildings can be grouped into four main categories:

1. Two-storey commercial row brickwork buildings (Figure 31a);
2. Large commercial and residential brickwork buildings either detached or built in rows (Figure 31b);
3. Churches (Figure 31c);
4. Other typologies of buildings with either public or industrial use, such as the old University colleges, old stonework mills, and so on (Figure 31d).

These categories fall in the classes D, E, F and G of the New Zealand's URM building stock as identified by Russell and Ingham (2008).

(a)

(b)

(c)

(d)

Figure 31 – Unreinforced masonry structures typologies: (a) two-story commercial row buildings with unidirectional timber floor structure and timber or steel roof structure. The layout is extremely regular, with one or two rooms per floor and openings on the front wall only. Whereas side and partition walls bear the horizontal structure the façade does not have a bearing function. Parapets at the top level and braced awnings are common features. (b) Large commercial and residential buildings; structural features are similar to those of smaller buildings although the quality of construction and detailing seems in general superior, with better connections between structural elements. (c) Churches are either brickwork or stonework with pitched timber roof supported by the side walls and timber portal frames. Frequently the gable is taller than the roof. (d) Other buildings with public or industrial use. These are often large buildings that have undergone a change of use during the years, but because of their importance and cultural value have been carefully upgraded and are maintained to a high standard. Stonework and good quality brickwork are the common materials. Chimneys and towers are a common feature.

From the observations gathered in the surveyed areas, URM structures appeared to be the most highly affected by the earthquake. A large proportion had already suffered some level of damage during the Darfield earthquake, September 2010; in some cases the presence of pre-existing damage was indeed highlighted by temporary strengthening and support structures, which might have partly reduced the effects of the Canterbury event. In a number of instances instead, due to the lack of information regarding the integrity of the structure prior to February earthquake it was difficult to determine from on-site survey the evolution of damage as a consequence of the two separate events. However, in the light of the observations collected for buildings with evident or known damage due to the Darfield quake, the authors believe that the February earthquake generally worsened the existing state of damage, leading to cracking of other elements and collapse of larger parts of structures following patterns consistent with the damage provoked by the previous earthquake.

A number of structural characteristics common to URM buildings and influential to the seismic response were identified and are presented in the following in relation to observed damage mechanisms.

1. Material quality. Churches and public buildings (Figure 32a) generally had multi-leaf or rubble cavity walls with an outer wythe of cut stone blocks, while the brickwork of the other types of URM structure was made of frogged fired bricks laid with lime mortar. It is likely that the majority of important buildings had undergone grouting, as stonework masonry displayed a good cohesion. Instead, the extended collapse of large portions of brick masonry panels was probably determined by the weakness of the bond deriving from the poor quality of mortar and possibly from the use of strong bricks with a vitrified superficial finish (Figure 32b–c).

(a) (b) (c)

Figure 32 – (a) Example of stonework of historic building. In brick walls, the poor quality mortar and weak bond of masonry units was evident from the complete separation of bricks and from extensive cracking in the mortar joints but hardly affecting the bricks in both (b) collapses of portions of walls and (c) punching failures of anchors.

2. Masonry fabric. Whereas a number of buildings were constructed with a regular bond with good through-thickness connection, the lack of internal connection observed in some multi-leaf and cavity walls provoked separation and out-of-plane failure of wall panels (Figure 33).

(a) (b)

Figure 33 – Evidence of damage as a consequence of the poor through-thickness connections of masonry walls: (a) complete lack of headers leading to separation of the two single-leaf panels of the wall; (b) small and far apart metallic profiles unable to ensure the unitary response of multi-leaf wall and prevent out-of-plane collapse of outer wythe.

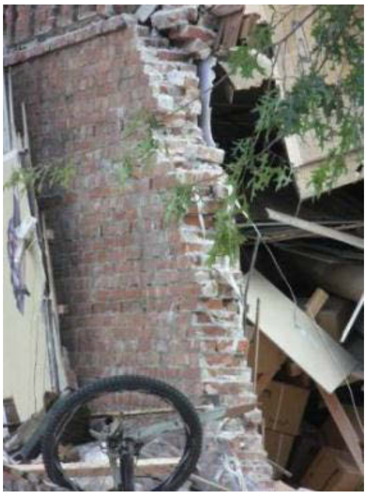
3. Quality of connections between structural elements. The most commonly observed damage mechanism involved the overturning of the front walls as consequence of the lack of effective connections to horizontal and other vertical elements (Figure 34a–c). In the case of both small commercial row buildings and churches, the vulnerability of vertical wall panels was aggravated by the fact that the horizontal structures only bear on the side walls, or on timber portal frames. Consequently, façades and gables behave like free standing elements with shear capacity reduced by the lack of horizontal friction that is otherwise determined by the weight of horizontal structures bearing onto the walls. The presence of better connections between the façade and other structural element shifted the damage mechanism to overturning involving portions of the side walls (Figure 34d–e) or arch mechanisms (Figure 34f). In-plane cracking was observed only in those cases where the quality of connections, these being either original or upgraded by strengthening, ensured a box-like behaviour of the structure and the transmission of the horizontal loads depending on the relative stiffness of the structural elements (Figure 34g–h).


(a)


(b)

(c)


(d)


(e)


(f)


(g)

(h)

Figure 34 – Observed correlation between the quality of connections and damage mechanisms. Overturning of (a) whole or (b) portion of the facade as consequence of poor connections between front walls and other vertical and horizontal elements. The vulnerability of vertical wall panels was worsened by the lack of vertical compression of horizontal structures, which could only bear on the side walls. Indeed the floor beams span in the direction parallel to the façade (c) and the roof trusses rest on the side walls (b). (d) Portions of side walls were involved in the overturning mechanism of the façade in case of better corner connections, like in the case of reinforced bed joints (e: detail of bed joint reinforcement). (f) Good corner connections and connections to floors with lack of connection to roof structure determined a horizontal arch mechanism involving only the top portion of the front wall. (g) X-shape cracking indicates a box-like behaviour that allows the transmission of horizontal loads in the various structural elements and the activation of the in-plane resistance. (h) In this case the box-like behaviour was achieved by the improvement of connections by regularly spaced anchors.

(a) (b) (c)

Figure 35 – Example of collapse of free standing elements such as: (a) parapets and (b) gables. (c) Retrofit by bracing, when present, prevented the collapse of elements.

4. Presence of free standing elements, such as parapets, gables and chimneys. The lack of strengthening of these elements caused recurring failures (Figure 35).

The vast majority of URM buildings had undergone upgrade and retrofit, yet not all the strengthening systems were able to achieve the expected level of performance during the earthquake. This under-performance was partly because of the quality of the existing materials, and partly because of shortfalls in the design and detailing of the strengthening. Recurring systems, with relative advantages and pitfalls, are briefly reviewed below:

1. Metallic rods and profiles had been extensively used to enhance the connection between wall panels and timber structural elements, these either being part of the horizontal structures, namely floor beams or roof trusses (Figure 36a), or of the inner timber frame in those buildings that feature outer masonry structure and inner structure with timber columns and portals. Anchors may have had an end plate, been embedded in the masonry by resin, or fixed by mechanical connection. Irregularly spaced, insufficiently sized and widely spaced apart elements proved ineffective in avoiding separation of structural elements (Figure 36b-c), whilst regular layouts prevented out-of-plane failures (Figure 36d). The poor quality mortar also affected the performance of anchors, causing premature bond failure in the mortar joints and thus compromising the transmission of the horizontal load between structural elements.

(a) (b)

(c) (d) (e)

Figure 36 – Metallic anchors: (a) example of application for connection of floor beam to wall. Note that the lack of connection in the other direction determined the out-of-plane failure of the façade. (b) Insufficient size and (c) ineffective positioning of the elements compromised the performance of connectors; yet (d) the better layout and size of elements proved effective in preventing damage in similar structures. (e) Failed steel girt and anchors.

2. Steel strapping. A number of steel strapped masonry structures were observed. The details on these structures were provided as a rapid repair and stabilisation system for the cracking that resulted from the Darfield event. As the ground motions were greater than the design ground motions, the performance of strapped structures can be deemed to be satisfactory (Figure 37a), although in the case of large structures (e.g. Figure 37b) the size of the steel elements seemed inadequate from a damage limitation point of view due to lack of compatibility between the axial deformation of the steel elements and the allowable drift of the façade.

(a) (b)

Figure 37 – (a) Example of steel strapped structure. (b) Cracking in side wall of church due to overturning of façade. The size of steel elements was not compatible with the allowable drift of the façade. Damage was not prevented, although collapse did not occur.

3. Steel bracing and steel frames (Figure 38a–b). Even if frames performed well and prevented the collapse of horizontal structures, the sizing and spacing of connections between braces and walls was insufficient to restrain the out-of-plane mechanism of masonry panels (Figure 38b–c). In fact, a better global performance is likely to be due to better detailing (Figure 38d).

4. Concrete frames (Figure 39) presented the same problems as steel frames: despite behaving well during the quake, they were not sufficiently connected to prevent out-of-plane damage to the masonry walls.

(a)

(b)

(c)

(d)

Figure 38 – Examples of (a) steel bracing and (b) additional steel frame. Although in some cases the retrofit by steel bracing proved highly successful (d), in others the bracing, despite preventing the collapse of the roof, could not prevent the overturning mechanism of masonry panels (c). This constitutes a major drawback both in terms of safety and, for heritage buildings, of loss of original material.

(a)

(b)

(c)

Figure 39 – Example of (a) additional concrete frame and (b) detail of the connection with the roof structure. The moment frame performed satisfactorily, but the lack of good connections of the masonry walls to the frame caused out-of-plane failure of the outer leaf of the wall (c).

Reinforced Masonry (RM) Buildings

The majority of observed reinforced masonry structures were two-storey commercial buildings, which performed well and did not suffer any major structural damage (Figure 40).

Damaged buildings suffered from bad construction practice issues, such as lack of reinforcement bars or grouting. However, this was mostly the case of 1960s and 1970s auxiliary structures or gravity bearing structures only, such as additions to older buildings or garages (Figure 41), and internal blockwork walls.

Figure 40 – Row of undamaged or lightly damaged RM buildings.

(a)

(b)

Figure 41 – 1970s RM addition. The building was badly damaged due to differential settlement in the ground. However, neither reinforcement bars, nor grout were to be found in the masonry.

A notable example of a heavily damaged commercial RM building is shown in Figure 42. The building was on an obtuse-angled street corner, and therefore the layout was almost triangular, comprising one very open façade (shown in Figure 42a) with two long RM walls along the other two sides. This led to a heavy concentration of demand at the front corner column, which appeared to be rather lightly reinforced (Figure 42b). The neighbouring building was also very close (but unconnected) and pounding damage was observed in both buildings (Figure 42c).

(a) (b) (c)

Figure 42 – Heavily damaged commercial RM building on Victoria Street. (a) Irregular layout led to (b) heavy concentration of demand on corner column. (c) Some damage due to pounding with adjacent building was also observed.

Special Seismic Buildings

As discussed in previous sections, New Zealand is a world leader in the use of isolation devices for the seismic protection of buildings and bridges. The first “modern” use of engineered seismic isolation is widely regarded to be the William Clayton building in Wellington, which sits on lead-rubber isolation bearings, invented by New Zealander, Bill Robinson. Modern research efforts in New Zealand engineering departments focus on “damage avoidance” strategies of seismic design, including post-tensioned jointed precast concrete moment frames and rocking walls, following on from the lessons of the US PRESSS programme (Priestley et al., 1999). As noted previously, design criteria for these systems were incorporated as a normative appendix in the most recent (2006) reinforced concrete standard.

Primarily due to its classification as a low-to-moderate seismic hazard in New Zealand loadings standards (since zonation maps were adopted in the 1965 version), Christchurch only has two prominent examples of buildings incorporating these technologies.

- The Christchurch Women’s Hospital, which is the only building in the South Island that is seismically isolated.
- The Southern Cross Hospital Endoscopy Building, which is understood to be one of only two buildings in New Zealand currently incorporating PRESSS technology (the other in Wellington; CCANZ, 2011).

The EEFIT team did not visit either of these hospitals, but were told by local engineers that structurally they performed well, and were fully functional after the earthquake (CCANZ, 2011).

It should be noted that hospitals would normally be considered “Importance Level 4” due to their post-disaster function. For this importance level, the table from which Table 5 was excerpted (PCFOG, 2009) gives a tolerable impact under the 1/500 ground motion described as “moderate”: building function affected for less than one hour; minor damage to structure; moderate damage to building fabric; contents affected; building accessible and safe to occupy. The tolerable impacts for the 1/1000 and 1/2500 ground motions are severe and very severe, respectively, for which the descriptions can be read from Table 5. Even in light of these enhanced performance requirements, the design intent of these special seismic systems appears to have been successfully met.

6. PERFORMANCE OF LIFELINES

Lifelines were particularly badly affected due to severe and widespread liquefaction (as described further in Section 7). Settlements of hundreds of millimetres were not unusual and lateral spreading of up to a metre was also often observed. The liquefaction and lateral spreading had a significant effect on buried services such as liquefaction induced settlements, tension cracking due to lateral spreading and floatation of services (Figure 43), and severed many lifelines. A programme of repairs was underway at the time of the survey and the majority of Christchurch had running water (although it was unsafe for drinking) sewerage and electricity; however, a few areas still did not have these essentials. While strictly speaking residential roads, and some of the services described in this sections are not lifelines, they have been included in this section as the wide scale disruption of these is worth reporting.

Figure 43 – A petrol station having suffered liquefaction. The underground storage tanks at this station have floated when the soil was in a liquefied state. The station was closed at time of survey.

Electrical Distribution Networks

Electricity was widely available at the time of survey; however in some areas power was supplied by connecting diesel generators to the transformers. In these areas rationing of electricity was in force and residents reported that blackouts were common. Many overhead power poles were leaning dramatically as a result of liquefaction. A number of transformers had suffered settlements due to liquefaction (Figure 44); however it could not be established if this had resulted in disruption of supply.

(a) (b) (c)

Figure 44 – (a) An electrical services box that had tilted due to liquefaction; it is not known if this resulted in loss of service. (b) A typical services failure that has been exposed ready for repair. This service was located approximately 200m from the river and was damaged from lateral spreading – note the crack in the road substrate in (c), this crack occurred across the entire road surface and is displayed in (b).

Water Supply

Water supply was badly affected, with most areas in liquefied regions without drinking water immediately after the earthquake. At time of survey most water was back online; however drinking water was still being transported to some areas.

Sewerage

The river was signed as "unsafe for human contact" and at times the odours coming from it seemed to confirm this. A number of pumping stations were not in operation and their contents were being pumped directly into the river. It could not be established if this was water supply or sewage, and, if it was sewage, whether the residents had been disconnected from the system so that raw sewage was being pumped into the river (see Figure 45).

(a) (b)

Figure 45 – (a) A pumping station pumping its contents into a storm water drain, and (b) a transportable pump, pumping directly from one manhole into the river.

Transport Networks

The continuity of transport links immediately after the earthquake was variable. Many bridges had suffered rotations of their abutments due to lateral spreading and had been closed for a few days while their safety was assessed. At the time of survey all bridges visited were open, but many had speed restrictions due to the differential settlements between the bridges (which suffered little or no settlement) and the approach roads (where settlements could be severe). Bridges are discussed further in the following sub-section.

There was also evidence of differential lateral movements of roads (especially near bridges) resulting in the buckling of the paved surface. Many road surfaces were very uneven due to liquefaction (Figure 46a) and a few roads were impassable due to lateral spreading (Figure 46b, Figure 47). At the time of survey nearly all the roads were operational; however there were many areas where the uneven nature of the road led to severe speed restrictions and many were restricted to residents only.

(a)

(b)

Figure 46 – (a) A residential road showing large undulations in the surface. This particular section of road had a speed restriction of 30 km/hr; however, for this particular section observed speeds were much lower than this. This level of pavement deformation was not unusual and large areas of Christchurch experienced this level of damage. (b) Road failure due to lateral spreading. The trees in the top right of the photo show the start of the bank down to the river. Note the fire hydrant (yellow in the bottom of the picture) showing that settlement had occurred along the service alignment. This was a common mechanism in the failure of roads and services due to lateral spreading.

Figure 47 – Stormwater outlet showing that large aggregate could be mobilised. The aggregate shown formed part of the sub-base of the road.

BRIDGES

At the time of survey all bridges visited were open; however, due to damage from the earthquake, many had speed restrictions and/or weight limits imposed. Damage to bridges due to ground shaking was minimal, with no bridges collapsing or significantly moving off their substructures due to ground shaking and was generally limited to spalling of concrete resulting from pounding, or movement of the superstructure on the substructure. There was some damage due to plastic hinges forming in columns and bridge piers. The greatest damage to bridges resulted from the widespread and acute lateral spreading experienced during the earthquake. This damage was generally confined to the abutments that suffered various degrees of rotation. Flow of the soil towards the river generated very large lateral forces. These forces pushed the abutments towards the river; however they were restrained at their base by the piles that they were founded on, resulting in rotation. It was not possible to determine how far the piles may have moved towards the river (if at all), however all the bridges inspected had plastic hinges at the tops of the piles and so they must have provided significant lateral resistance. The lateral spreading also resulted in settlement of the approach road. The mechanism for this was that the abutments, which were on piles, do not settle vertically; however as the soil spreads towards the river there was a corresponding loss from the embankments causing the approach roads to settle. This mechanism was common and was the main reason for speed restrictions and failure of the services in the bridges.

(a)

(b)

(c)

(d)

Figure 48 – Fitzgerald Ave Bridges (built in 1964) showing (a) rotation of the abutments, (b) plastic hinging forming in the top of the piles connected to the abutments (not the reinforcing bars connecting the abutments to the pier, (c) settlement of the approach road due to lateral spreading and concrete spalling due to pounding and (d) failure of services again due to lateral spreading. These bridges were not significantly damaged in the Darfield earthquake.

(a)

(b)

Figure 49 – (a) Abutment failure of the Anzac Drive bridge (built in 2000), caused by lateral spreading This bridge was founded on steel piles, embedded well into the abutment, and it is assumed that plastic hinges formed in the tops of these piles. (b) Spalling of the bottom of the bridge deck at the abutments.

(a)

(b)

(c)

(d)

(e)

(f)

Figure 50 – (a) Gayhurst Road bridge. This bridge was open at the time of survey, but liquefaction under the approach roads and lateral spreading had resulted in (b) large ramps to access the bridge. (c) The deck itself was relatively undamaged, but compatibility between the abutments and the main deck resulted in damage to the wing walls of the abutments as well as severing of services. (d) The base of the abutments had moved inwards resulting in lateral displacement of the central pier and formation of a plastic hinge approximately two thirds of the way up this pier. (e) and (f) The detailing of the connections between the water main under the bridge and the bridge deck enabled the pipe to slide and although some lateral buckling of the pipe did occur it did not fail. At the time of writing, this bridge has been reported as having collapsed in the June 13 aftershock.

(a) (b)

(c) (d)

Figure 51 – (a) Damage to an arch footbridge near Snell Place. This bridge contained one of the main high voltage cables servicing approximately 20,000 homes. The lateral movement of the embankments led to a hogging hinge at midspan shown in (a) and (b). (c) shows the crack on the top of the bridge at the location of the hinge, (d) show and a sagging hinge at the abutment.

Figure 52 – Steel bridge over Avon River in CBD, showing buckling of outer arch due to movement of abutments resulting from lateral spreading. Although this damage looks significant, the structural system consists of steel girders under the main deck and arches under the footpaths. There was no obvious damage to the main girders.

(a)

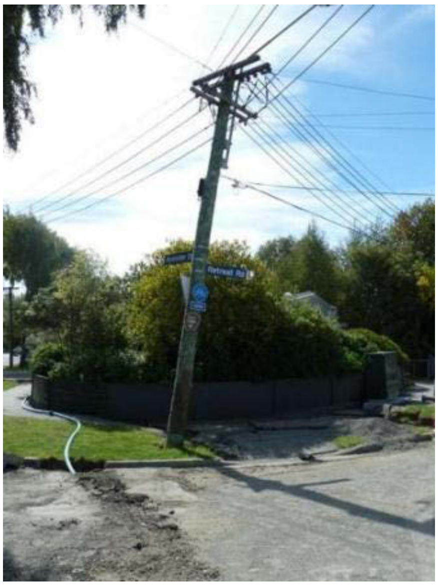
(b)

Figure 53 – Albert Street bridge. The superstructure of this bridge showed no obvious signs of damage, but the abutments had suffered significant rotations (a) resulting in the bridge becoming detached from its elastomeric bearings and at time of survey the bridge deck was bearing on hardwood wedges. (b) Plastic hinges were visible at the tops of the piles as was closely spaced helically wound lateral reinforcement. The bridge had been closed for approximately three days while its safety was assessed and then reopened. Speed restrictions were in place due to settlement of the approach spans. A main service (presumably water supply) had failed and had been repaired.

Figure 54 – Moorhouse Avenue overpass, which consisted of many piers (of which two failed). At this particular location the pier consisted of a double column on either side of an expansion joint. The piers were relatively slender about the axis perpendicular to the bridge deck and the pier on one side of the expansion joint formed two plastic hinges about its weak axis due lateral movement of the bridge deck and vertical load. The supporting structure shown in the image was intended to protect the bridge from further damage due to aftershocks rather than to enable reopening. The bridge was still closed at time of survey and was responsible for significant traffic disruption.

Telecommunication

At the time of survey all telecommunications visited were operational; however, many telegraph poles were severely out of plumb and switching boxes had suffered settlements (see Figure 55).



(a)

(b)

(c)

(d)

Figure 55 – (a) and (b) out of plumb telecommunication poles were common in areas of liquefaction, as was (c) liquefaction around telecommunications boxes. As the boxes were founded on robust concrete foundations, settlement and rotations of these were usually minimal; however they were often accompanied by greater local settlement of the ground due to liquefaction. It is not known if this significant differential settlement severed the underground services. (d) Repair of buried services was commonly observed at the time of survey and this picture depicts a telecommunications service being repaired

7. GEOTECHNICAL ASPECTS

FOUNDATION PERFORMANCE

This section gives an overview of the observed performance of building foundations following the February 22nd earthquake. The focus is restricted to buildings located in areas with liquefiable soils – i.e. areas of observed ground failure – as it is assumed that in non-liquefied areas structural damage was solely due to ground shaking. Following the Darfield earthquake it was reported that damage due to ground failure occurred primarily in residential suburbs (Cubrinovski et al, 2010); however, the February 22nd event caused ground failure again in residential areas and also in some areas of the CBD. As a result, the relative performance of both low-rise residential buildings and modern multi-storey buildings could be assessed in this survey.

The EEFIT team had several opportunities to observe examples of ground failure (for example Figure 56) and corresponding building response during the course of the mission. Damage to single and double storey buildings was observed in the residential damage surveys (see Section 5 and Appendix A). Some cases of multi-storey building damage were noted through by the EEFIT team members in the on-going structural assessments carried out in the CBD. Further to what was observed independently during the EEFIT mission, work was carried out for several days with a research group at the University of Canterbury (coordinated by Assoc. Professor Cubrinovski), investigating a number of interesting case studies of multi-storey buildings within the CBD. Several of the specific cases cited in the following section were observed in conjunction with the work of the UC research group; further information on these buildings can be found by referring to (Giorgini et al., 2011).

At the time of this survey, limited information was available concerning foundation types in the city – particularly of multi-storey buildings – other than what could be directly observed in the field, and often from the exterior of buildings only. Information regarding the ground failure in individual cases was dtermined from observations of signs of liquefaction, ground deformation, lateral spreading, and the performance of adjacent buildings in a given area. Assumptions were supplemented, where possible, by reports from local engineers or academics familiar with the area. Due to limited building access, little can be said of the structural performance of the building resulting form to ground failure, aside from the obvious impacts highlighted herein.

Multi-Storey Buildings

As discussed in Section 5, most multi-storey buildings in Christchurch are located in the CBD. The CBD consists of a collection of building types, ranging from low-rise commercial buildings to high-rise office buildings of up to approximately 20 storeys. Most low- and medium-rise buildings are thought to be situated on shallow foundations such as spread footings, mat foundations, or shallow basements. Some higher rise buildings were observed to have deep basements, while a few buildings were identified as being pile-supported.

Due to the severe liquefaction in the northern section of the CBD, the chosen case-studies were concentrated in the area inclusive of Armagh St to the south and Salisbury St to the north. This is also where other instances of poor foundation response were noted by the EEFIT team during structural inspections. The described area contains a good cross-section of buildings types, including a variety of multi-storey buildings as described above, as well as several residential complexes. Some blocks within this region were affected by lateral spreading towards the river, while others showed evidence of a more localized slumping. Further research into the historical geological make-up of the area could help verify observations regarding these localized areas.

(a)

(b)

Figure 56 – (a) Lateral spreading cracks near Oxford Tce and Colombo St. (b) Evidence of liquefaction on Salisbury St.

Shallow Foundations

The majority of buildings in the CBD are situated on shallow foundations. These would typically be expected to perform poorly if the depth of foundation did not extend beyond the depth of the liquefiable layer. Given that alluvial surface deposits prone to liquefaction exist in the region at shallow depths of up to 15 to 20 m (Cubrinovski et al; 2010), it is not surprising that many foundations did suffer some damage. The extent of this damage was varied, however, with many buildings observed to have minor to moderate damage, and a smaller number suffering more severe failure. These failures could be attributed to ground movement due to liquefaction, in both vertical and horizontal directions. Vertical movement was the prevalent mechanism observed within the zone of study, and the few cases of lateral movement tended to occur in conjunction with some vertical settlement.

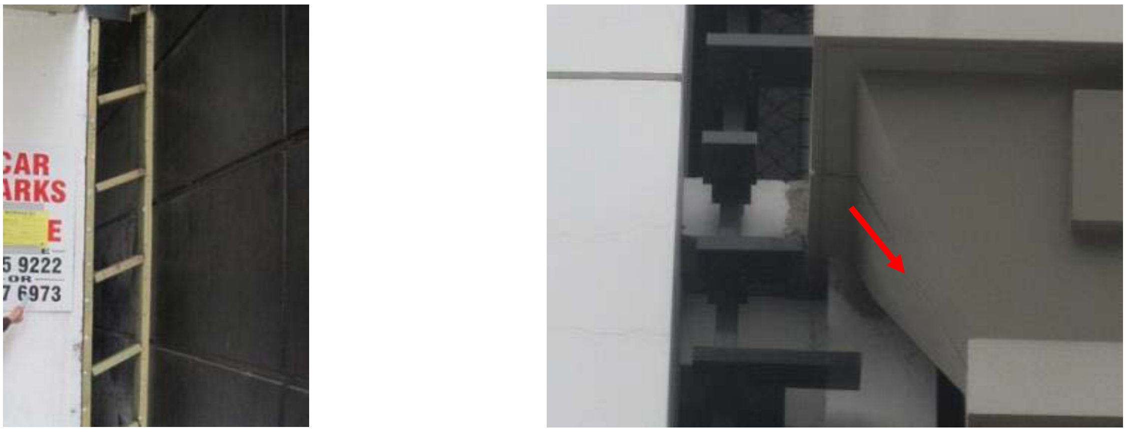
Vertical Movement

Vertical movement of the ground, typically in the form of settlement, could be attributed in large part to volume loss in the soil as a result of liquefaction. This can be safely assumed based on the amount of sand ejected (and hence removed) from the ground in the form of sand boils, as well as voids observed in areas of ground deformation (see Figure 57). Vertical movement of this nature was found to cause three main building responses: settlement of the building with the ground, settlement of the building relative to the ground, and settlement of the ground relative to the building. In this instance, the first two modes constitute a probable negative response, while the third may be an indication of good performance of the foundation. However, often it is difficult to determine the global response of the building based on these localized relative movements. Observations on all three mechanisms are provided in the following:

Figure 57 – Ground deformation on Madras St exposing void beneath the pavement.

Settlement of buildings with the ground

Settlement of shallow founded buildings with the ground was widely observed where signs of severe liquefaction were present. Indications of this type of settlement were not observed in relative movement of the ground and building, but rather in fixed components where the movement of the building can be measured, or in comparison to adjacent buildings that had not settled. Examples of such indicators are shown in Figure 58.



(a) (b)

Figure 58 – (a) Wooden struts illustrating relative movement between two buildings (assumed to be horizontal initially). (b) Movement of one building relative to another. The former position of the component of the right can be seen in the lack of paint on the left building.

Non-uniform slumping of the ground may result in differential settlement of the building, which often manifests in an observed tilt. Several buildings in the CBD were thought to be tilting slightly, though most were estimated to be one degree or less. Figure 59 shows a building with apparent differential settlement at one end. In this example, little structural damage is evident from the exterior.

(a)

(b)

Figure 59 – (a) Building observed to have settled with the ground at one end. (b) Tilt relative to adjacent building. On this particular street, features such as sand boils, pavement compression, voids, and damage to buried services were concentrated near one corner of the building, indicating a localized area of slump (right side of part a).

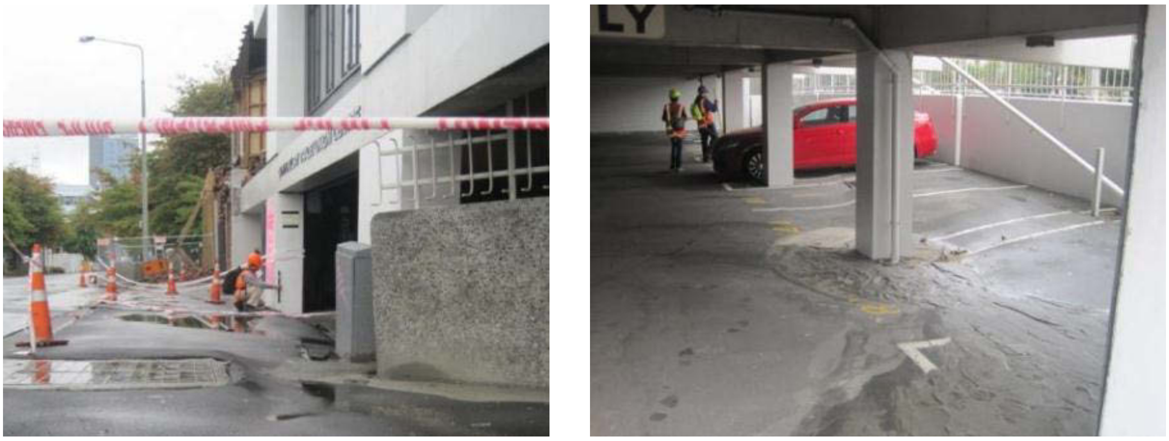
In some cases, building components such as entranceway slabs were observed to settle more than the main structure. This indicates that the foundation of the component was not tied together with that of the main structure. In Figure 60 it can be seen that some damage occurred at the building interface as a result of this settlement. In general, however, many buildings which were seen to settle with the ground – differentially or otherwise – often appeared to act in a rigid manner. Thus, little of the overall damage to the structure could be attributed to the ground failure. In fact, many of these buildings did not appear from the exterior to have suffered excessive structural damage.

(a) (b)

Figure 60 – (a) Entranceway to building breaking away from main structure. (b) Close-up of separation between entranceway and building.

Settlement of buildings relative to the ground

Another observed response in liquefied areas was further settlement of buildings with respect to the ground. Accompanying bulging of the ground outside the building, or heave of floor slabs in the interior, indicated a possible bearing failure of the soil underneath the foundation, as a result of strength loss due to liquefaction (Bray and Stewart, 2000). Examples of these indicators in both the exterior and interior of two buildings are shown in Figure 61 and Figure 62.



(a) (b)

Figure 61 – (a) Bulging of the pavement in front of the columns of a 6 storey building reported to be on spread footings. The non-structural wall had also settled relative to the ground. (b) Heave/deformation of the pavement slab in the ground floor parking lot. Sand observed at column locations.

(a) (b)

Figure 62 – (a) Cracking in the ground floor slab of a 5 storey building. (b) 75 mm settlement of a column relative to the level of the bulging floor slab.

Of the examples observed, bearing failure beneath the foundations seemed to occur predominantly in buildings with high aspect ratios, or with foundations consisting of separate footings (Figure 61). This makes intuitive sense considering that these buildings would subject the soil to a larger bearing pressure than other foundation types.

In addition to ground-floor damage due to settlement of the building core, damage was noted in some structural components, such as shear cracks in multiple beams near the beam-column joints. This damage, along with the apparent differential settlement of the columns, could be an indication of a more flexible response of the building. More research is needed, however, to confirm the relationship between the observed damage, ground failure, and type of foundation used.

Settlement of the ground relative to buildings

Two examples of high-rise buildings reportedly on shallow foundations, and that appeared to have perform well were observed,. In these cases, the ground was seen to settle with respect to the building – in some locations by up to 200 mm – as seen in Figure 63. While no information was available regarding the type of foundation of the first building, the second was reported to have a raft foundation on a non-uniform gravel layer (M. Cubrinovski, personal communication, 2011). Despite this apparent lack of settlement, it was suspected at the time of survey that the building was tilting slightly. Interestingly, the apparent tilt was away from the river, in the direction of less observed liquefaction. Although more sophisticated measurement is needed to confirm the tilt in this building, this example demonstrates how relative movement of the ground may be somewhat misleading when assessing foundation performance.

(a) (b)

Figure 63 – (a) and (b) Localized settlement of the ground relative to a 20 storey high-rise building on shallow foundations. This type of settlement was observed on all sides of the building.

Lateral Movement

When the ground is sloping – towards a river, for example – liquefied soil will tend to flow in the direction of lower ground. This may cause spreading at the ground surface and, if a stiff crust exists above the layer, lead to lateral pressures being exerted on buried structures. Within the CBD, a few multi-storey structures were observed to be affected, not only by vertical settlement, but also this lateral ground movement. In one case, lateral spreading towards the river resulted in a 4 storey building frame being pulled apart at several points, from the foundation to the roof (Figure 64). It was possible to attribute this damage to ground spreading, as tension cracks appeared in the road roughly corresponding to the cracks in the building and similar cracks were present in a masonry building across the street. In this instance, it would appear that the strength of the building and foundation (at these joint locations) was not sufficient for it to sustain the lateral pressures and act as a rigid unit.

(a) (b) (c)

Figure 64 – (a) and (b) Gaps in a 4 storey building frame due to lateral spreading of the ground. (c) Tension cracks in the road and vertical cracks in masonry building across the street. This building was located near a bend in the river, and likely experienced forces (and displacements) in several directions. Settlement relative to the ground was noted at one end, consistent with the widening of the cracks near the top of the building.

Another example of lateral movement was observed in a building that had experienced localized settlement at one end (shown previously in Figure 59). At the opposite end from this settlement, a gap was found in the pavement next to the building, indicating a lateral displacement of the same magnitude as the settlement (Figure 65). In contrast to the building in Figure 64, this building has remained rigid and intact through its translation.

(a) (b) (c)

Figure 65 – (a) and (b) 15 cm void at one end of a building, indicating lateral movement of the building. (c) Tension cracks in the pavement at a central column.

Buildings on Pile Foundations

At the time of survey, only two of the observed buildings could be confirmed to have piled foundations, though a handful of others may have had this type of foundation. The performance of these two buildings was mixed. One 6 storey building, in an area of very severe liquefaction, but little evident lateral spreading, was shown to suffer minimal settlement compared to the surrounding ground. Figure 66 shows the foundation beams of the building exposed by almost 300 mm, trapping the cars parked inside. Despite this dramatic settlement of the ground, the building was thought to have a slight tilt, which could be due to a number of factors. This again must be confirmed with more accurate measurement.

(a) (b)

Figure 66 – (a) and (b) Exposed foundation beams in a building with piled foundations.

Another observed building was reported to be on piles, though the depth of these piles was thought to be varying, with some possibly quite shallow (M. Cubrinovski, personal communication, 2011). This building was one of two high-rise buildings quite obviously titling relative to one another. In the N-S direction, the buildings were tilting away from each other, while in the E-W direction they were tilting towards each other such that they made contact at roughly the third storey. From the front of the building in question, a slight bulging of the pavement suggests that the piled building had settled relative to the ground, which is consistent with the direction of tilt. The neighbouring building, with a basement foundation, appeared to have settled with the ground but not further. Limited external damage was observed in both buildings. This is an interesting case, as the impact of lateral spreading (which was observed nearby) as well as a possible interaction between the responses of the two buildings must be considered. Further research into this case, and several other multi-storey buildings described above, is being conducted by the UC research (Giorgini et al., 2011).

(a)

(b)

Figure 67 - (a) Two multi-storey buildings tilting relative to one another. The taller building is thought to rest on piles of varying lengths. (b) Slight bulging of the pavement near the columns of the piled building.

Low-Rise Commercial and Residential Buildings

Most residential and low-rise commercial buildings in the Canterbury region are supported on shallow foundations. It was thought that a small handful of residential buildings were on piled foundations, but the EEFIT team was not able to observe such buildings in the survey. Most foundations in this category consisted of either a simple slab on grade, or continuous perimeter footings with few residential houses in Christchurch having basements.

Due to widespread liquefaction in the suburbs, hundreds of thousands of homes were reported to be affected by ground failure. Many of these houses will likely be deemed beyond economic repair in upcoming inspections, according to insurance representatives. Suburbs that experienced liquefaction in the Darfield Earthquake, such as Bexley, experienced further liquefaction in the February 22nd event. Detailed visits to some of these suburbs were beyond the scope of the EEFIT damage surveys; however, it can be assumed that similar damage reported in the first event (Cubrinovski et al., 2010) would have occurred again if the same areas re-liquefied.

In the areas surveyed by EEFIT, foundations of residential buildings were found to have failed in much the same manner as the shallow foundations described for multi-storey buildings – that is, by settlement, tilting, and lateral spreading. Movement of foundations was also observed as a result of complex landslide movements in the Port Hills. Whilst some vertical movement was common in these structures, lateral ground movement (such as that shown in Figure 68) often appeared to be the cause of more detrimental damage to the structure.

Figure 68 – Lateral spreading in the back yard of a house near the Bexley Wetlands

Vertical Movement

Numerous cases of minor foundation cracking concentrated around airbricks were observed, caused by differential settlement of the concrete slab or footing. In a representative example shown in Figure 69, extensive liquefaction has occurred around the building perimeter, and it appears to have settled relative to the ground. This was confirmed by an internal inspection, where damage to the timber floor boards could be seen (Figure 69c). Multiple cracks seen on the exterior foundation and walls were also observed in the building interior. It was reported by an insurance representative that in general, timber floors performed better than concrete floors in houses that experienced settlement of this nature.

(a) (b) (c)

Figure 69 – (a) and (b) Liquefaction and foundation damage of a single storey office in the CBD. (c) A pipe protruding from the timber floor boards, confirming settlement of the building.

Lateral Movement

Lateral displacement of concrete slab foundations or concrete/brick ring foundations (with suspended floor joists) was observed in several single storey timber residential buildings, with movement of up to 300 mm in the direction of ground movement. In some cases the building moved as a unit, while in others the timber frame had moved independently from the foundation, indicating inadequate connections between the timber frame and slab foundation (see Figure 70). The impact of lateral spreading in the building was evident in the movement of part of the concrete slab foundation, which has split underneath the timber structure and forced movement of the entire frame. The main timber structure itself appeared to have sustained only minor damage.

(a)

(b)

Figure 70 – (a) and (b) Single storey dwelling situated approximately 20 m from the river bank in the Dallington/Avonside district. Lateral movement of 100 mm towards the river was observed in this foundation.

In other observed cases, the main building frame remained in contact with the foundation, but could not withstand the lateral forces induced by the earthquake, resulting in separation of the components. Figure 71 shows examples of masonry and timber buildings that both appeared to have separated in this manner.

(a)

(b)

Figure 71 – (a) A residential masonry building in Dallington which has experienced separation due to lateral spreading. (b) Separation of a timber frame house in Avonside, also due to lateral spreading.

LANDSLIDES

The earthquake triggered several types of ground movements including lateral spreading and landslides. A landslide is defined here as the outward and/or downward movement of soil or rock. Landslide hazards were generally created by a combination of steep slopes and weathered and fractured volcanic rocks and soft soils, and these hazards were triggered by the extremely strong ground shaking produced by the 4 September 2010 and 22 February 2011 earthquakes. Landslide types included rock falls and collapse of rock slopes and cliffs, localized shallow landslides involving retaining walls and landscaped ground, cracks and rents, and deep-seated landslides involving rock slopes. Landslides were generally limited to within 5 km of the epicentre in the Port Hills and Lyttelton areas. The extent of the landslides recorded by the responsive GeoNet/GNS landslide survey is shown in Figure 72 and the location and type of landslides (at 07/02/2011) is shown in Figure 73. Landslides triggered by the 22 February earthquake mainly affected an area of around 30 km^2 and possibly up to 150 km^2, based on early estimates from Hancox et al., 2011.

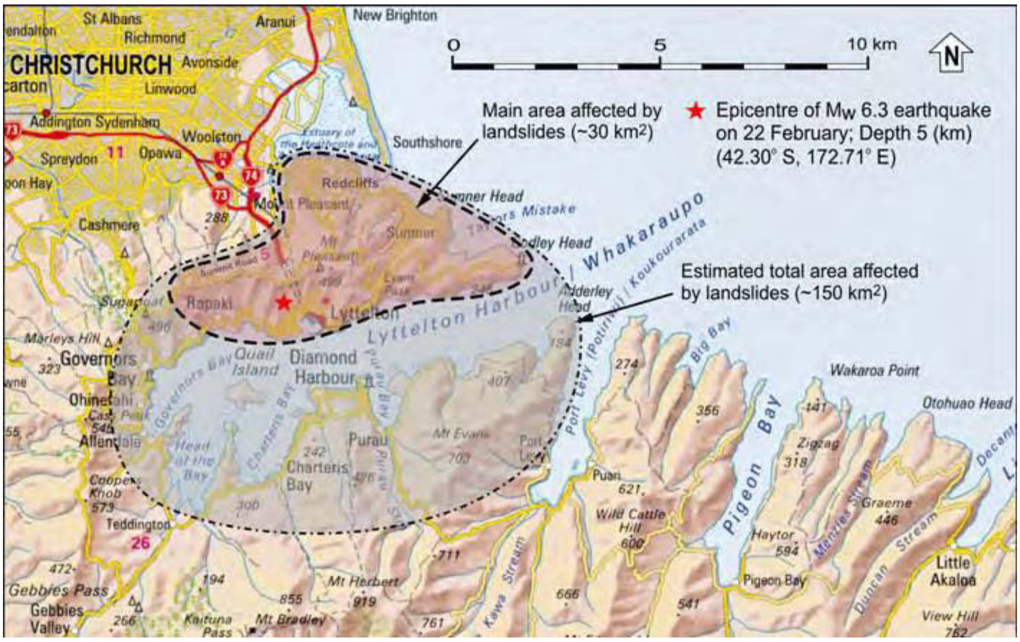


Figure 72 – Map showing area affected by landslides triggered by 22nd February, 2011 earthquake (Source: GNS/Hancox et al., 2011).

The Port Hills rise rapidly from a rocky coastline to 499 m at Tauhinukorokio/Mt Pleasant, and are the flanks of an eroded extinct basaltic volcano (6–10 million years old) that has been uplifted and incised by drainage channels to form steep slopes. Thick gently-dipping beds of lava (lava flows) form near-continuous bluffs that can be traced around the hill sides. The undercutting of these more erosion-resistant beds over time by sapping erosion frees the blocks of basalt that fall and accumulates to form a mantel of talus. The flanks of the volcano have been eroded by coastal erosion throughout the last 10,000 years, during periods of high relative sea-level. The formation of cliffs has resulted in over-steepened inland slopes that encircle flat low-lying town of Sumner on the northern flanks. In places, the abandoned sea cliffs have been quarried for rock fill material which was used for reclamation or ground improvement of poor ground conditions in areas recently inundated by the sea such as at Moncks Bay and Sumner. In several places this quarrying has resulted in vertical or overhanging slopes and may have left slopes more susceptible to landslides.

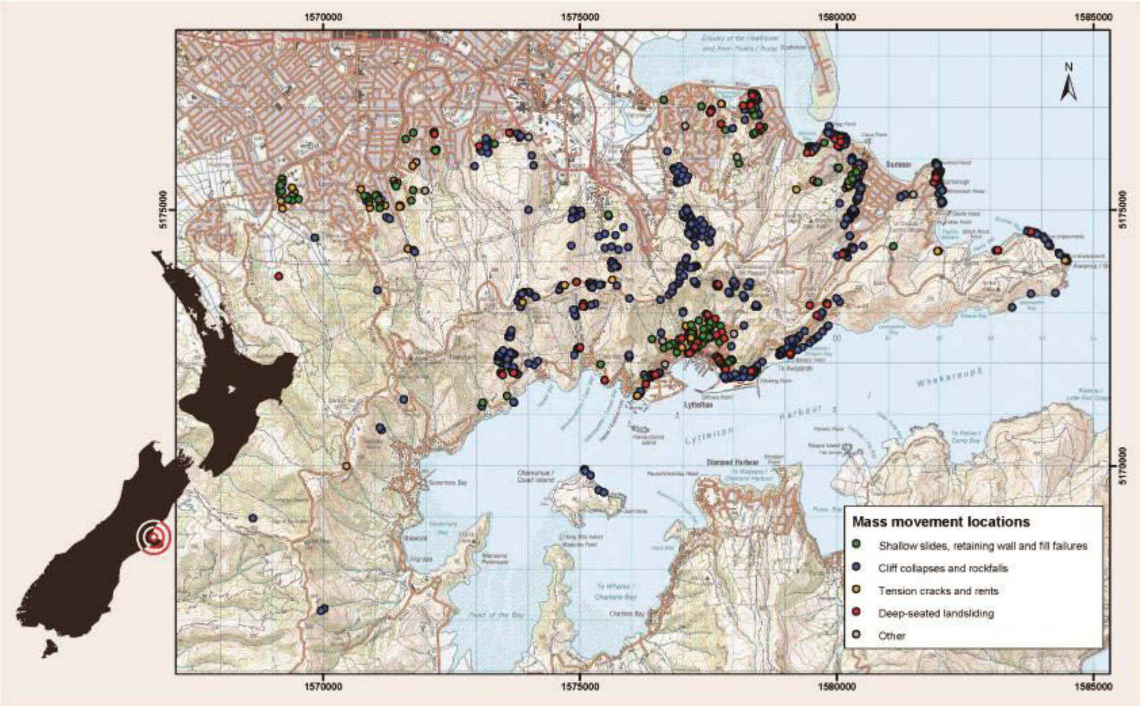


Figure 73 – Map showing distribution of mass movements in the Port Hills area following 22nd February, 2011 earthquake. (Source: GeoNet/GNS)

Responsive landslide mapping

The day after the earthquake GeoNet/GNS undertook an aerial reconnaissance helicopter flight over the area to identify the worst affected areas. The findings are reported by Hancox et al., 2011 and many of the oblique aerial photographs taken by G Hancox, GNS Science, have been reproduced here with permission. Two days after the earthquake a series of high-resolution stereo colour aerial photographs were taken by New Zealand Aerial Mapping; these images provide full ground coverage of the affected areas. Much of the ground damage can be identified on the photos and could be compared with other historic imagery taken before and after the 2010 Darfield earthquake. Three days after the event a ground team including GeoNet/GNS Science and University of Canterbury staff undertook a ground-truthing exercise to identify and map slope stability issues more fully.

Geotechnical database

Reports of geotechnical issues were recorded in a geotechnical database which included location, description, mechanism, potential effects, initiation, treatment plan, history and owner and contractor contact information. The slope stability field inspection data was initially managed in a central GIS by GNS staff, and then later handed over to volunteers at the Emergency Operations Centre (EOC). The landslide hazard maps were updated daily in the weeks after the event to enable emergency managers to monitor progress and assign work to contractors.

Hazard and risk assessment

The data compiled in the geotechnical database were used to undertake a semi-quantitative risk assessment that involved rating each geotechnical issue, including landslides, in terms of 'potential effects' or hazard and likelihood of an 'initiation mechanism'. The product of the two hazard ratings was described as 'Hazard Exposure rating'. This semi-quantitative risk assessment provided decision makers a subjective judgment of how vulnerable each receptor was to a range of foreseeable trigger mechanisms. The scheme was strongly focused towards assessing immediate risk, but does not attempt to rate or distinguish different levels of hazard. The highest scores received for 'potential effects' (n = 50) were obtained where a slope stability issue posed a threat to lifeline routes or highly vulnerable infrastructure (e.g. reservoirs). The timely collection and provision of the geotechnical information, storage in a digital database, and display in a GIS environment enabled the emergency response teams, decision makers and contractors to have access to up-to-date information to make management decisions and prioritize resources. An advantage of the GIS based scheme was that it provided a spatially based 'live' risk register that could be updated daily, presented on a map, and could be transferred to other GIS users. One limitation of producing digital datasets daily in a rapidly evolving situation was that multiple file versions could be confusing to users and so careful versioning and release procedures are required.

Observed landslide damage

The EEFIT team (DB and AM) inspected several sites in the Port Hills area on 17th and 18th March 2011; the location of sites is shown in Figure 74. The EEFIT team was accompanied by engineering geologists Dr Phil Glassey (GNS), Dr Lis Bowman (University of Canterbury), and students from the University of Canterbury.

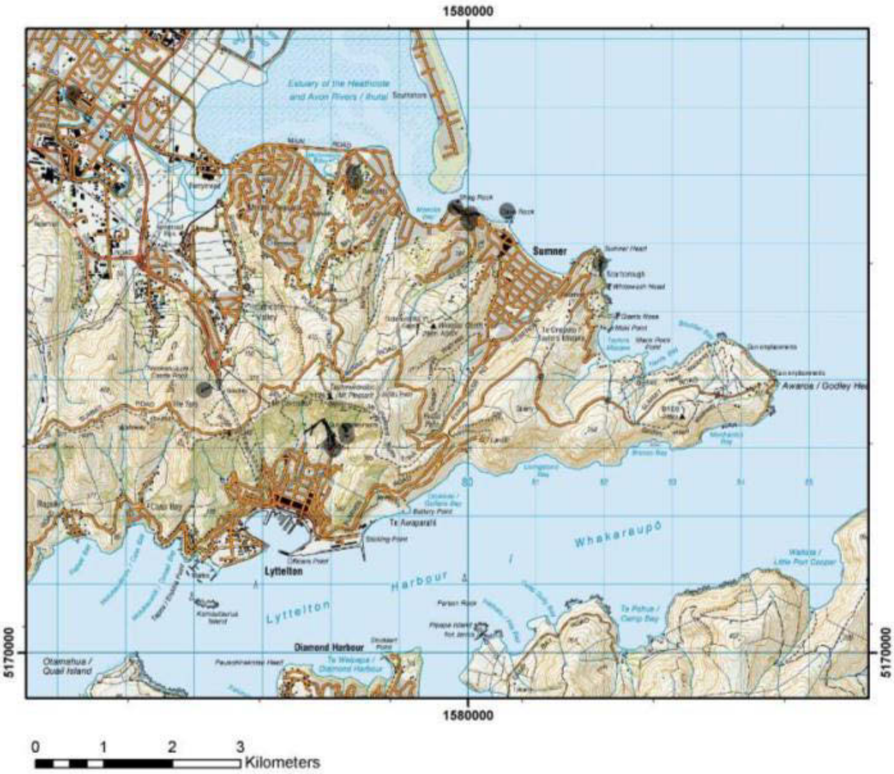


Figure 74 – Map showing the location of sites visited by EEFIT landslide team as Black dots.

The landslide types observed in the Port Hills included: rock falls, boulder rolls and bounces, deep-seated rock slope failures, earth falls that developed into earth flows (loess) and shallow rotational failures in fill materials. Many cliff tops have extensive tension cracks, more typical of deeper seated rock slope failures. Landslides affected lifelines routes such as roads and houses. The tunnel through to Lyttelton was not seriously affected although some minor rock falls around the tunnel portal were recorded during both the September 4th 2010 and February 22nd 2011 earthquakes. At least five fatalities resulted from rock falls including three in the Sumner-Redcliffs area and two walking in the Port Hills foot tracks (Hancox et al., 2011). The landslide sites visited by EEFIT are described in the following section, and are listed by locality.

Lyttelton/Mount Pleasant: rock rolls, bounces and potential debris flows

Many rock falls and boulder rolls and bounces were triggered by both the September 2010 and February 2011 earthquake and so interpretation of the age of rockfalls required gaining some knowledge of both events from local engineers. Above the port town of Lyttelton several large bounders rolled a distance of up to 500 m down through scrubland towards the town. The boulders made tracks through the hill slope vegetation, shown in Figure 75, and dents in the soil and road tarmac at Hyllton Heights Rd provide evidence that boulders rolled down the hill. There were often being funnelled into gullies. One large basalt boulder of approximate dimensions of 3 m^3 (approximately 6 t) was seen in a residential garden below Hyllton Heights Rd. Many of the boulders that started to roll during the earthquake were sitting perched on the slope before the earthquake. At Hyllton Heights Rd, boulders had bounced on the road, making an impact dent in the road top, before coming to rest in a garden.

Large quantities of rock fall debris were liberated from basalt cliffs above Littleton, as shown in Figure 76, and this supply of debris was accumulating in gullies above the town. Theseg rock fall debris could increase the debris flow hazard to the town below. At the time of the EEFIT visit local engineers were discussing temporary engineering solutions to reduce the debris flow hazard. One option being considered was the construction of gabion catch dams filled with the locally sourced rock debris.

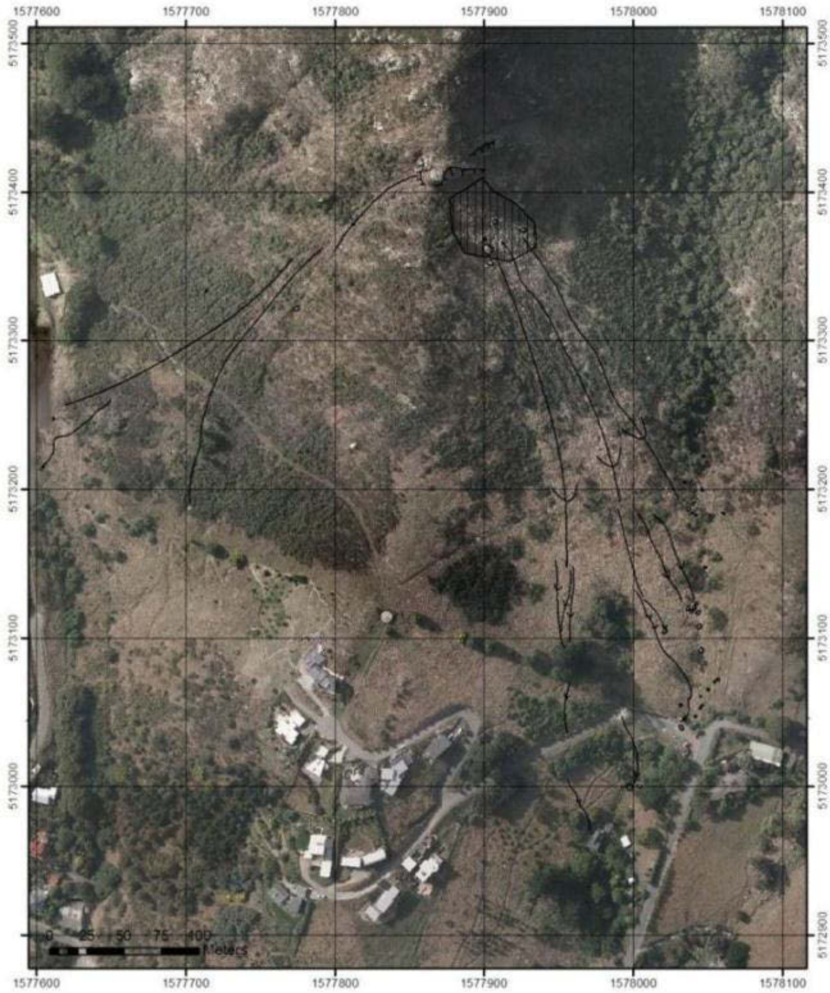


Figure 75 – Above the port town of Lyttelton strong ground shaking mobilised perched boulders. The boulders carved tracks through the vegetation and their tracks, picked out in black, are visible on this air photograph taken by New Zealand Aerial Mapping on 24th February, 2011. Reference grid is NZTM.

Figure 76 – Rock falls from a 20m high basalt scarp generated high volumes of rock debris that was accumulating in the steep drainage gullies above Lyttelton.

Sumner/Sumner Head: deep-seated topple in sea cliff

A network of ground cracks had developed along the top of the 80 m high cliffs at Sumner Head. The cracks that can be traced on the air photo shown in Figure 77 persist for a distance of around 100 m along the top of the cliff. Close inspection of these ground cracks, shown in Figure 78, concluded they were dilated at the surface by up to 200 mm and were persistent to at least 3 m below surface, and their occurrence is controlled by the bedrock joint pattern of vertical cooling joints (columnar type). Additional narrower cracks were found up to 30 m back from the cliff in the gardens east of Searidge Lane. The cracks indicate the onset of a deep-seated rock slope failure with a toppling type failure mechanism.

Figure 77 – Tension cracks affected the grass and path along the top of the 80 m high sea cliff at Sumner Head in the eastern Port Hills. NZTM 1581943, 5175709. Photo taken by Graham Hancox/GNS on 24 Febuary 2011.

Figure 78 – Deep tension cracks at Sumner Head probably caused by dilation of vertical joints in basalt lava flow. The ground to the right of the crack being inspected was part of a potentially unstable topple block. Taken from NZTM 1581945, 5175712 facing northeast.

Moncks Bay/Main Road: rock falls

The 60 m high cliff above Main Rd was the source of multiple rock falls that mostly landed on the flat ground and at the base of the slope. At the eastern and western end several boulders of up to 2 m^3 rolled a distance of up to 40m onto Main Rd, partly blocking the road as can be seen in the aerial photograph in Figure 79. Empty cargo containers were placed at the base of the cliff to temporarily shield the road from further rock fall as shown in Figure 80. The geology of the slope is jointed basaltic (hawaiite) to trachytic lava flows interbedded with tuff and breccias. The cliff was formed by sea erosion during a period of higher relative sea-level and the cliff profile has been modified by subsequent quarrying.

Figure 79 – Aerial photograph taken on 24/02/2011 showing the unstable cliffs above Main Road in Sumner. The hatching shows the area affected by rockfalls. The collapsed sea-stack, locally known as Shag Rock, is indicated offshore. Ground cracks affecting the road and gardens along Kinsey Terrace are also traced in black. Reference grid is NZTM.

Figure 80 – Cargo containers temporarily placed between the cliff and Main Road to intercept any further falling debris. The rock fall debris obstructed the west bound (left) lane closing it to traffic.

Sea stack collapse: rock topple

A sea stack, known as Shag Rock, collapsed during the 2011 earthquake. The extent of the collapsed debris, interpreted from the air photograph, is shown on Figure 79. The upper half of the pillar was a lava flow composed of vertically columnar jointed basalt and is shown after collapse in Figure 81. The pillar was laterally unsupported allowing the rock mass to dilate during the strong horizontal and vertical shaking. The block sizes generated were controlled by the spacing between joints and bedding.

Figure 81 – Collapsed sea stack, locally known as Shag Rock.

Kinsey Terrace: deep-seated landslides

Fissures in the road tarmac at Kinsey Tce above the cliffs were observed. Wide tension cracks in the lawn of one house, shown in Figure 82, aligned with those on the road. The house in Figure 83 has come off its ring foundation probably as a result of the landslide related ground movement. A continuous GPS station was deployed by GNS/GeoNet within a few days after the event to monitor slope movement. The logger had a remote connection that could send near-real time data back to the office (within an hour) for specialist interpretation and warning of significant ground movement.

Figure 82 – a wide roughly north-south aligned tension crack (left) opened up in a landscaped garden south of Kinsey Terrace. A continuous GPS station seen in the far right of image was deployed by GeoNet/GNS Science, three days after the event and was sending near-real time slope movement data back to the office for interpretation by experts.

Figure 83 – At this house in Kinsey Terrace the concrete ring foundation had separated from the frame structure due to landslide related ground deformation. Cracks in the garden also occurred and can be seen in the mid-ground

Redcliffs/ Glendevere Terrace: rock falls and deep seated rock

The formerly quarried slopes at Redcliffs are approximately 50 m high and stand at around 80 degrees. They are composed of well-jointed basaltic (hawaiite) to trachytic lava flows interbedded with less pervasively jointed tuffs and breccias (including lahars), which are cut by dykes and minor lava domes. The bedrock was capped by at least 3 m of loess deposits typically composed of yellow-brown windblown silt deposits, combined with fine sand or clay. Earthquake induced rock falls impacted properties built close to the base of the slope. An aerial photo of the site is shown in Figure 84. The rock bund built at the back of the school, shown in Figure 85, caught most of the rockfall debris; however, as a result, the bund was full to capacity, and so another fall might overrun the bund and reach the school grounds. At the top of the cliffs wide slope parallel tension cracks affected gardens and properties, and indicate dilation of the rock mass and possible further movement of the slope (see Figure 86 and Figure 87). Rock falls landing on flat, grassed, generally horizontal ground travelled a horizontal distance approximately equal to the slope height, but occasionally as large as 1.5 times the slope height.

At Glendevere Tce, approximately 45 m back from the cliff edge (NZTM 1578358, 5176919) a 50 m long network of fissures were aligned roughly parallel with the ridge crest was observed. One interpretation of the feature is a ridge-rent, a failure of intact rock that provides evidence for topographic amplification. An alternative interpretation is a landslide tension crack, which, as the cliff is approximately 50 m high would indicate initiation of a deep seated rock slope failure with a failure surface of 45 degrees. Another interpretation of these fissures is that they could be related to lateral spreading. The presence of sand at the surface (most likely derived from loess mantling the slope) provides evidence for liquefaction during shaking. During the visit a continuous GPS unit had been located on the potential failure wedge by GeoNet/GNS to monitor any ground movement.

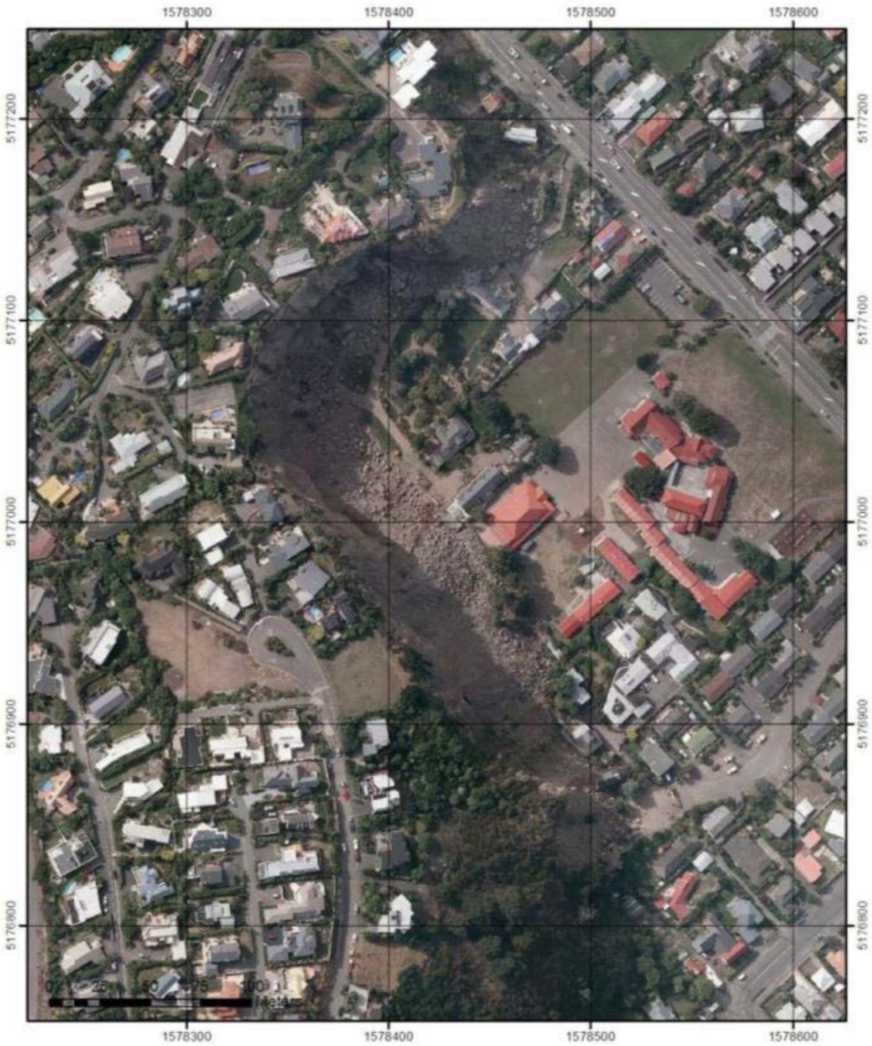


Figure 84 – At Redcliffs a rockfall from the quarried volcanic cliffs caused damage to buildings below, including a school. Cracks had developed along the cliff top and caused damage to roads, houses and gardens. (Credit: New Zealand Aerial Mapping)

Figure 85 – Large rock falls from the quarried former sea-cliff at Redcliffs affected Redcliffs School and Moa Cave Guest House. The houses along the ridge on top of the slope in the right of the image experienced very intense ground shaking. (Taken by G Hancox, GNS Science)

Figure 86 – An alternating bed of jointed basalt, tuff and breccia failed along the former quarry slopes at Redcliffs. Cracks in the cliff top left houses clinging to the edge of the cliff top and others buried by rockfall debris.

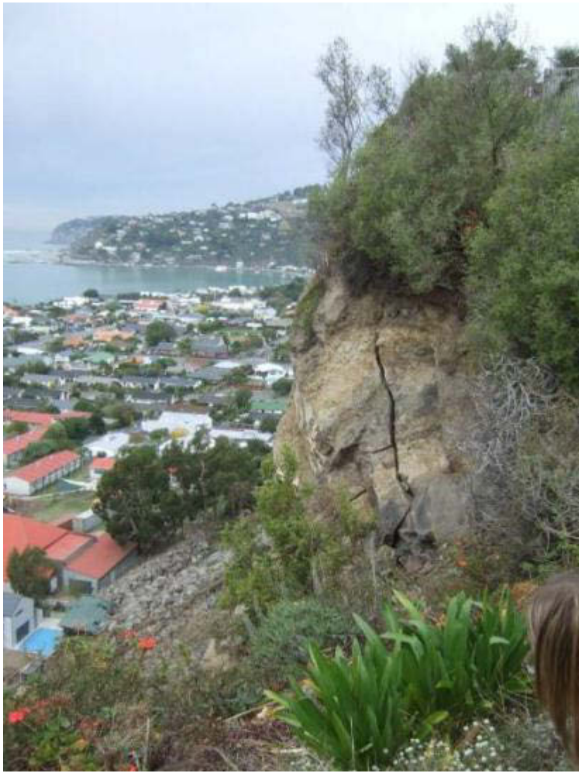


Figure 87 – Cracks along the tops of cliffs at Redcliffs.

Glendevere Terrace: Retaining wall performance

Two soil retaining wall located at Glendevere Tce, exhibited very different performance during the earthquake. The 2.5 m high gravel filled gravity crib wall had deformed and showed a residual overhang, however the 2.5 m high wooden pile retaining seen in the right of Figure 88 had suffered no visible damage. Small failures of low (<1.5 m) unreinforced rock retaining walls and soil slopes were frequently observed across the Port Hills area.

Figure 88 – Earth retaining walls at Glendevere Terrace. The wooden crib wall (far ground) suffered deformation, while the flexible wood pile retainer (near ground) suffered no obvious damage.

Wakefield Avenue: rock falls and bounce

Large rockfalls impacted properties on the corner of Wakefield Ave and Nayland St causing at were reported to have caused at least three fatalities. The vertical cliff above the RSA building was the source of rock falls that disintegrated upon impact with the ground and debris bounced across Wakefield Ave for up to 60 m, stopping at 27 Wakefield Ave. The cliff above Wakefield Ave was approximately 30 m high and composed of thick beds of jointed basalt and interbedded tuff. Schmidt hardness tests were performed on fallen boulders, giving estimates of the intact unconfined compressive strength (UCS) of the basalt as 100±40 MPa, and the interbedded tuff was 30±10 MPa. The 20 m-long near-planar vertical joint forms the slope and this joint probably formed the rear failure surface of the 15 m long block that landed next to the RSA building, shown in Figure 90. The joint was stained red and this discoloration suggests the wall of the joint was weathered and pre-weakened prior to failure. The rock falls at Wakefield Ave were from previously quarried over-steepened slopes along a rock promontory and it was possible that topographic amplification could have contributed to the strength of shaking here. The clay tiled roofs of the houses along the ridge accessed by Richmond Hill Rd were more badly damaged; than the tiled roofs on the flat land below the slopes. This again indicates that the ridge experienced stronger shaking and provides evidence for topographic amplification effects.

Figure 89 – Rock falls caused damage and loss of life at the corner of Wakefield Ave and Nayland St. (Credit: G Hancox, GNS Science).

Figure 90 – A 15 m long boulder of tuff toppled from the quarried slope above the RSA building at the northern end of Wakefield Ave. The rock fragmented upon impact with the ground and some boulders bounced across Wakefield Ave for a distance of 60 m, twice the height of the slope.

Rapaki: rock rolls and bounces

Boulders rolled and bounced up to 600 m down the grassed slopes at Rapaki affecting proprieties along Governors House Rd. A large boulder bounced through the house shown in Figure 91.

Figure 91 – A house at Governors Bay Rd, in Rapaki was hit by a boulder that had bounced 600 m from a bluff to the north west. (Credit: G Hancox, GNS Science).

Slope monitoring methods

A range of slope monitoring techniques had been deployed to monitor landslide related ground deformations. Low cost, rapidly deployed slope monitoring techniques were being used by local geotechnical consultants to monitor unstable slopes. At several sites there had been crack monitoring pins (long nails) installed with brightly coloured tape stretched between pins to provide visual indications of large displacements. Wall cracks had been marked with paint and measurements with dates were listed to keep a running record. Several continuous GPS stations were erected to provide high precision near-real time observations of ground movements at the highest risk failure localities. Some of the highest rock slopes with extensive tension cracks developed at the top, such as the slope along Main Rd and at Redrocks, had been recently surveyed in detail using a long-range Terrestrial Laser Scanner shown in Figure 92. This method provides a high-accuracy baseline topographic model of the slope that can be compared against other data sets, such as the GPS data, to identify and monitor changes in slope morphology that might indicate the onset of progressive slope failure.

Lessons learnt

1. Emergency damage assessment teams require multiple fields of expertise. The team should include search and rescue personnel, structural engineers and geotechnical engineers or engineering geologists. This team structure should help to ensure that geotechnical issues, such as slope stability, which that could pose an immediate threat to lifelines or people, are identified at an early stage of the emergency response process.
2. Emergency damage assessment pro-formas should require geotechnical hazards to be documented in addition to structural hazards.

3. The collection of high resolution (low cloud cover) aerial photography immediately after an earthquake greatly assists in the identification of slope stability hazards, damage, and their impact on lifelines.
4. Slope monitoring techniques for emergencies need to be low tech, cheap to install and designed to allow repeatable measurements. More specialized and high accuracy methods, such as continuous GPS, can be used in critical areas.
5. Field data needs to be managed centrally and stored in an accessible format (e.g. GIS shapefiles) that can be used by consultants and contractors as the situation transfers from emergency response to recovery/repair mode.
6. The earthquake has generated several longer-term slope stability issues: (a) increased debris flow hazard, (b) mechanical weakening of slope materials leaves them more susceptible to instability, and exposed to the agents of weathering. Secondary failures could be triggered by future aftershocks or rain storms.
7. Gravity retaining wall structures that are not tied back and can dilate, such as loose gravel filled crib walls, offer little support during strong earthquake shaking, whereas well-constructed wooden pile walls may perform adequately.
8. The development areas susceptible to landslides within seismically active areas should consider the potential for slope stability failures above and below the site.

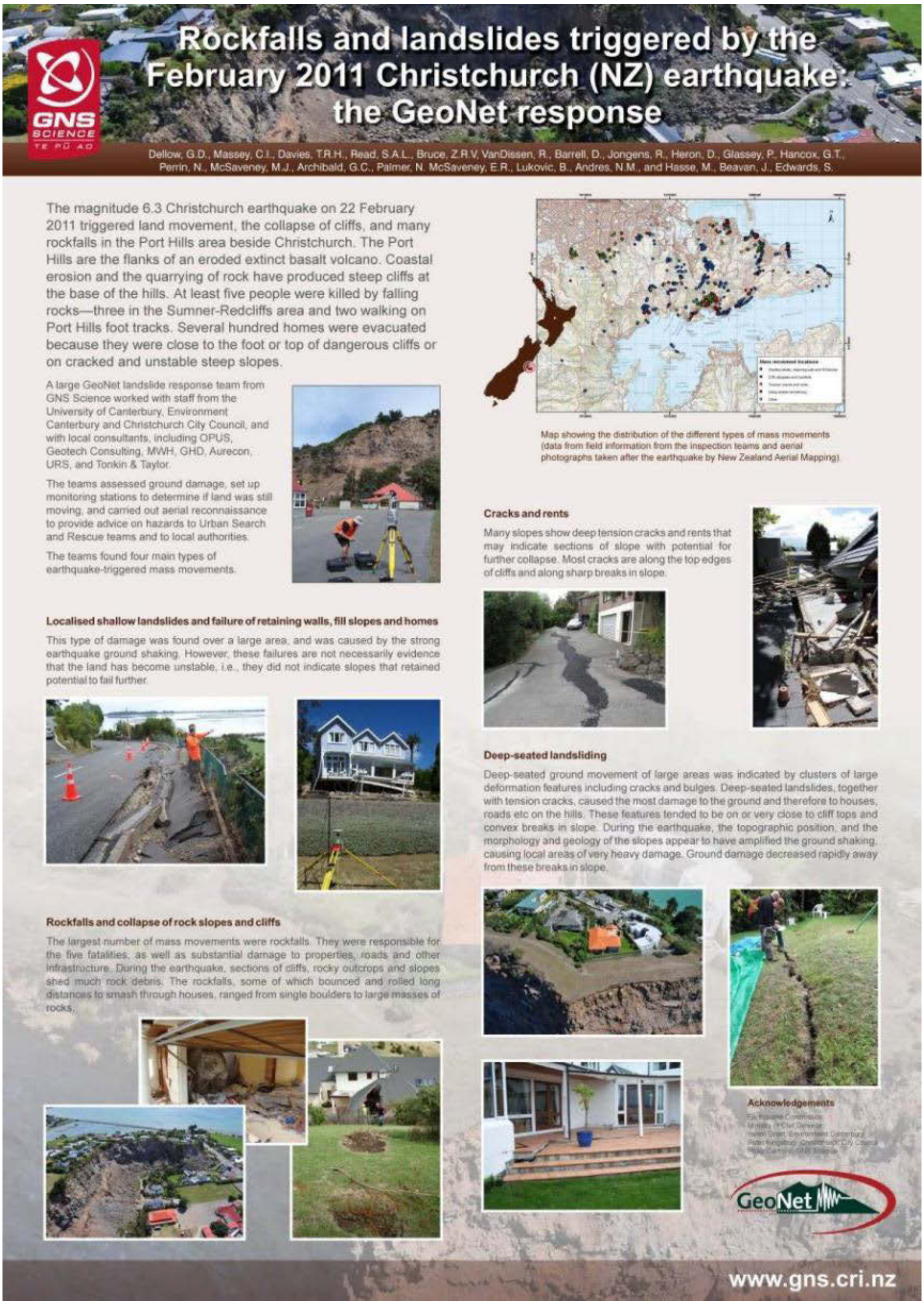


Figure 92 – Summary of GeoNet response to rockfalls and landslides triggered by the 22 February 2011 earthquake.

LIQUEFACTION

Liquefaction was a major feature of this earthquake and affected many engineered structures, geotechnical structures and lifelines. In particular lifelines (transport corridors and buried services) were badly affected, but also many structures experienced severe differential settlement of their foundations. Detailed descriptions of the impact of this liquefaction on the city of Christchurch are contained in the relevant sections within this report, while a brief description of the nature and distribution of liquefaction is given here.

The near surface geology of Christchurch is described in Section 2 and is briefly summarised here. The geology of the Christchurch area comprises predominantly recent Holocene alluvial gravel, sand and silt of the Springston Formation, with Christchurch Formation sediments mapped along the eastern part of the city. Bedrock is at a depth of approximately 600 m to 800 m. The Springston Formation alluvial deposits can be divided into overbank deposits of sand and silt (which are potentially liquefiable) and river flood channels that contain alluvial gravel (which are generally not liquefiable). The Christchurch Formation units are described as fixed and semi fixed dunes, and beach sands (generally less to not liquefiable). The groundwater table affecting the upper 10 m to 20 m of sediments is generally between 2 m to 3 m below the ground surface in the west and 0 m to 2 m below the ground surface in the central and eastern areas. The distribution of the potentially liquefiable deposits and the generally high groundwater table governed the distribution of liquefaction across the city.

Loose granular soils, such as the alluvial sands and silts beneath Christchurch, will generally densify when shaken. This densification, or compaction, results in ground settlement and is sometimes referred to as shake-down settlement. No liquefaction is required for this to occur.

Where the granular soils have low permeability, such as silts and sands, and are located below the water table, the groundwater between the soil particles becomes pressurised as the soil tries to densify and compact. The densification causes a rapid build-up of pore water pressure. This pressure forces the grains apart (reduces the effective stress) and causes the soils to liquefy. As earthquake shaking and ground oscillation continues, the ground is subject to both compression (causing compression ridges at the surface) and tension (causing tension cracks from the surface to depth). In many parts of Christchurch, compression ridges and tension cracks were visible in paved areas, such as roads carparks, (see Figure 93). Cracking of the ground provides a pathway for the liquefied, and pressurised, silt and sand to be ejected to the ground surface as sand boils or sand volcanoes. Such deposits of ejected sand and silt on the ground surface were widespread across Christchurch (see Figure 94).

Ground surface settlement must occur after ejection of sand and water to the ground surface and localised densification within the liquefiable ground. The settlement continues to occur until excess pore water pressures have dissipated in the liquefied ground. This ongoing settlement can be expected to continue for several weeks after an earthquake.

Figure 93 Compression ridges and tension Cracks

a

b

c

d

Figure 94 Examples of Sand boils

Liquefaction can also result in the flotation of buried services and structures, such as pipe utilities, manholes and tanks (see Figure 95).

Figure 95 Floatation of a petrol storage tank

In addition to ground settlement, liquefaction resulted in lateral spreading, where there was a lack of lateral confinement and a gradient. The lack of confinement was usually provided a stream or river bank or manmade slope. The ground moved laterally towards the free face with the liquefied layer providing low shearing resistance. Where the overlying ground was cohesive, this lateral movement resulted in a series of tension cracks or fissures parallel to the water body or manmade slope. In parts of the Christchurch this lateral spreading was measured at more than 3 m at the 'free edge' with tension cracks or fissures evident up to 400 m from the river. Very severe damage to land and buildings due to lateral spreading was observed in residential and commercial areas adjacent to the Avon River and to the Avon-Heathcote Estuary.

Due to time constraints a comprehensive survey of liquefaction was not possible; however it the worst cases and greatest extent of liquefaction appeared to have occurred near the Avon River. This distribution of liquefaction was most likely due to soils near the river being saturated, while those away from the river were less likely to be saturated (see lifelines and transport sections).

Lateral spreading was also significant especially in regions near the river. Lateral spreading was related to liquefaction; however, differs in that it occurs in soils already experiencing shear stresses (such as in slopes). In the case of liquefaction, the pore water pressure increases and the soil experiences a corresponding decrease in its effective stress. When this effective stress reaches zero the shear strength also becomes zero and the soil liquefies. In the case of slopes and other geotechnical features already under some form of shear stress, the effective stress does not need to reach zero, before the soil will experience a shear failure and so the soil may not liquefy before failure. In the case of this earthquake, large fissures were observed with no obvious slope visible. These fissures occurred because the slope that was spreading laterally was actually the river bank which was sometimes located hundreds of metres away.

As discussed in the geotechnical section of this report liquefaction induced settlements of up to 400 mm were not uncommon.

The earthquake spawned thousands of sand boils and much of Christchurch was covered in silt. As part of the disaster response of New Zealand a cleanup operation was quickly initiated and at the time of survey silt deposits had been cleared from all roads and other areas where they interfered with daily life; however many classic examples of sand boils could still be observed in parks and cordoned off areas (Figure 94).

The development of Christchurch on this former swamp land is revealed in suburb and road names throughout the city and an example is given in Figure 96.

Figure 96 Place and street names reflect that Christchurch was built on low-lying swampy ground

The liquefaction potential of the soils beneath Christchurch was widely known prior to the earthquakes and had been the subject of detailed regional studies. Further research on the subject was ongoing. Liquefaction hazard maps prepared by Environment Canterbury (ECan, 2004) and a number of previous similar studies indicate that large areas of ground beneath Christchurch had the potential to liquefy in a large earthquake event (see Figure 97).

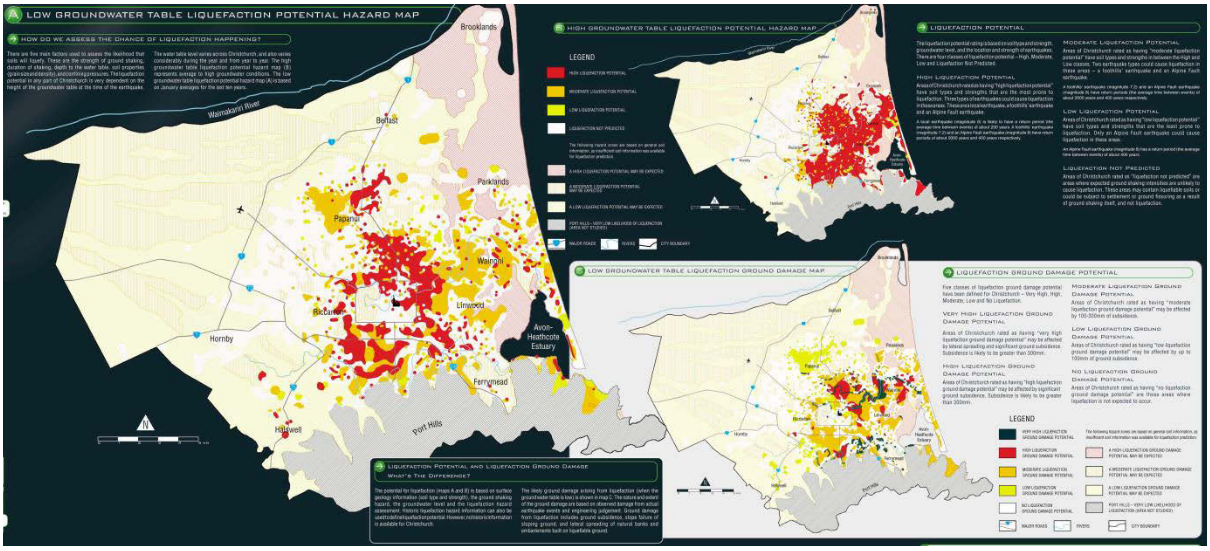


Figure 97 Liquefaction potential hazard maps (from ECan, 2004)

8. DISASTER MANAGEMENT

On 23rd February, 2011, the New Zealand Government declared a State of National Emergency applying specifically to the area under jurisdiction of Christchurch City Council, enabling powers as shown below:

> The CDEM Act 2002 provides for local authority delegated representatives, Mayors or the Minister to declare a state of local emergency. The Minister may declare a state of national emergency. Declared emergencies have a seven-day duration and may be extended or terminated.
>
> Emergency powers under the CDEM Act 2002 enable CDEM Groups and controllers to:
> - close/restrict access to roads/public places
> - remove/secure dangerous structures and materials
> - provide rescue, first aid, food, shelter etc
> - conserve essential supplies and regulate traffic
> - dispose of dead persons and animals
> - advise the public
> - provide equipment
> - enter onto premises
> - evacuate premises/places
> - remove vehicles
> - requisition equipment/materials and assistance
>
> Source: http://www.civildefence.govt.nz/memwebsite.nsf/wpg_URL/For-the-CDEM-Sector-CDEM-Act-2002-Index?OpenDocument

The State of National Emergency was extended on a weekly basis out to 30th April 2011 while vehicle recovery, structural assessment and demolition continued in some areas of the Central Business District (CBD). During this time the National Crisis Management Centre in Wellington was responsible for the overall response. The Ministry for Civil Defence and Emergency Management coordinated logistical support and reported to the Government.

During the response and recovery phases the Civil Defence Emergency Management Group (CDEM) operated out of the Emergency Operations Centre, a temporary facility housed in the Christchurch Arts Centre providing interagency coordination and emergency management in both response and recovery phases.

Fatalities and casualties

As at 7th April 2011, 182 people are confirmed to have died as a result of the 22nd February earthquake, with around half of this number being New Zealand citizens.

At least 95 deaths occurred in the collapse of the CTV building on Madras Street which housed the offices of Canterbury TV and the King's Education language school – many of the foreign victims were studying at this school at the time of the event (The Press, 2011b). Other locations of high numbers of fatalities in the CBD were the Pyne Gould Corporation building (building collapse), various locations on Manchester Street, and 2 buses on Colombo Street (due falling debris from building facade). Deaths in Christchurch suburbs were due to falling contents, building facades and rock falls.

As at 28th April 2011, New Zealand's Accident Compensation Corporation (ACC) reported 7,666 claims making the earthquake the largest mass injury event in the corporation's 37 year existence (Cairns, 2011). ACC provides no-fault personal injury cover for all residents and visitors to New Zealand. New Zealand residents are also eligible for death benefit grants. The injury claims included 5242 soft tissue injuries, 1108 cuts, 484 fractures and dislocations, 131 dental injuries, 63 concussion, 48 burns, 10 hernias, 7 amputations, and 2 trauma-induced hearing losses. ACC expects the lifetime cost of claims resulting from the 22nd February earthquake to approach NZ$200m.

Emergency Response

CDEM agencies involved in the response operated under the New Zealand Coordinated Incident Management System (CIMS). CIMS is based on the US Incident Command System (ICS) and is similar to the Australian version, Australasian Inter-Service Incident Management System (AIIMS) (MCDEM, 2006). The design facilitates coordinated a collaborative response of multiple international agencies. It is generic to any type of crisis or emergency and regulates the chain of command in an adaptable way depending on the agencies present.

A situation report indicated the scale of the local and national response to the earthquake. CDEM organisations activated following the earthquake included the Canterbury CDEM Group Emergency Co-ordination Centre (ECC), the Christchurch City Emergency Operation Centre (EOC), the Christchurch City Council (and surrounding district councils) Environment Canterbury (ECAN), New Zealand Police, the New Zealand Fire Service (NZFS), all three New Zealand Urban Search And Rescue (USAR) teams (Christchurch, Palmerston North and Auckland), and New Zealand Response Teams (MCDEM, 2011). Organisations such as Red Cross, Salvation Army and St John's Ambulance also quickly mobilised. Welfare centres were established to accommodate displaced people. The buildings of some agencies involved in the response were severely damaged or compromised by neighbouring buildings in the CBD, forcing them to rapidly relocate.

The involvement of 1400 members of the NZ Defence Forces and 116 soldiers from the Singapore Army constituted the largest military deployment ever on New Zealand soil. Deployment included medic teams, navy divers, engineers and NZRAF in a multitude of diverse roles (Dunne, 2011). The army and police controlled a cordon established to prevent people from entering for their safety and to prevent looting. Initially the cordon extended across the entire Central City area within 'the four avenues' – Bealey Ave, Rolleston Ave, Moorhouse Ave and Fitzgerald Ave.

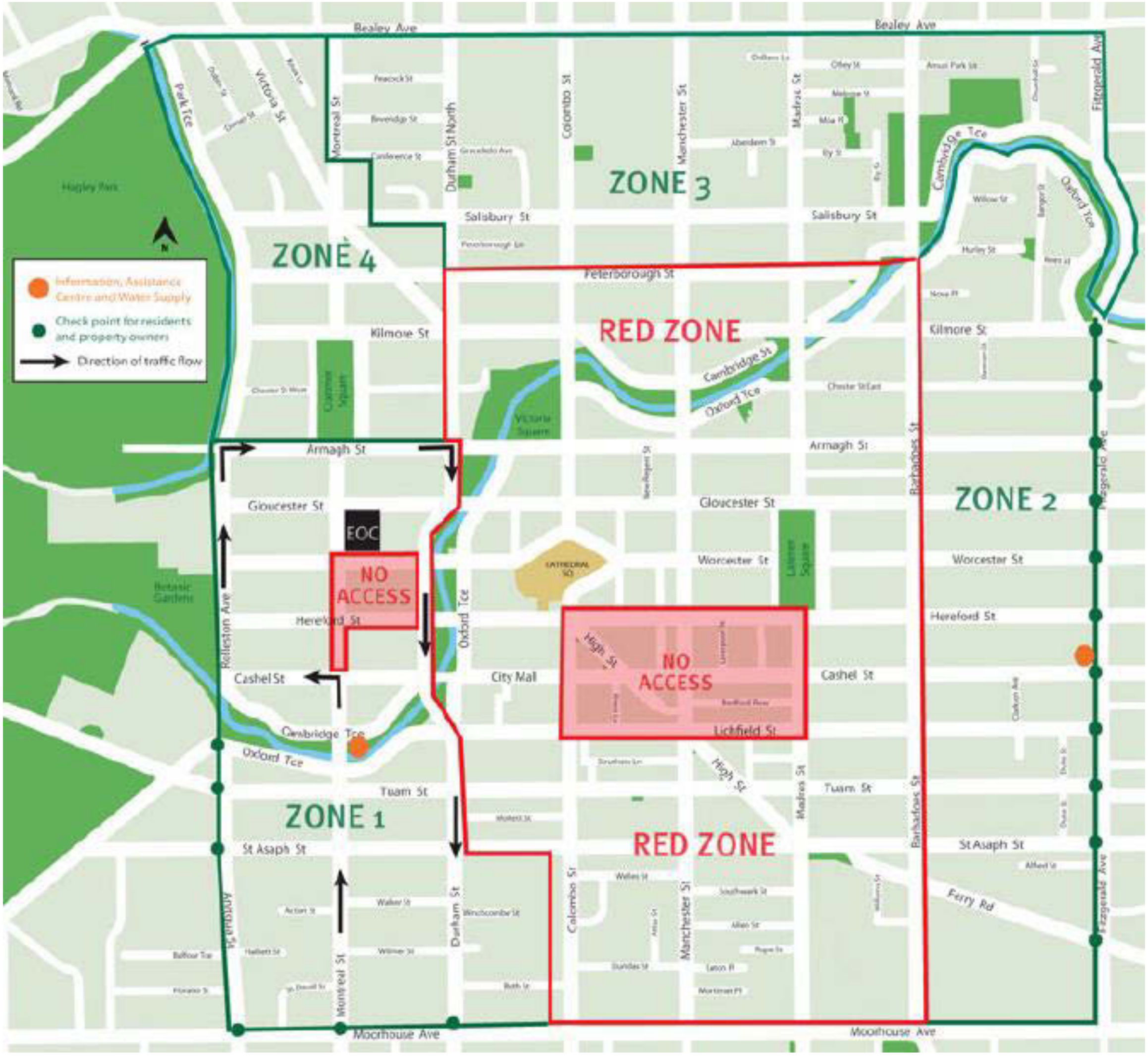
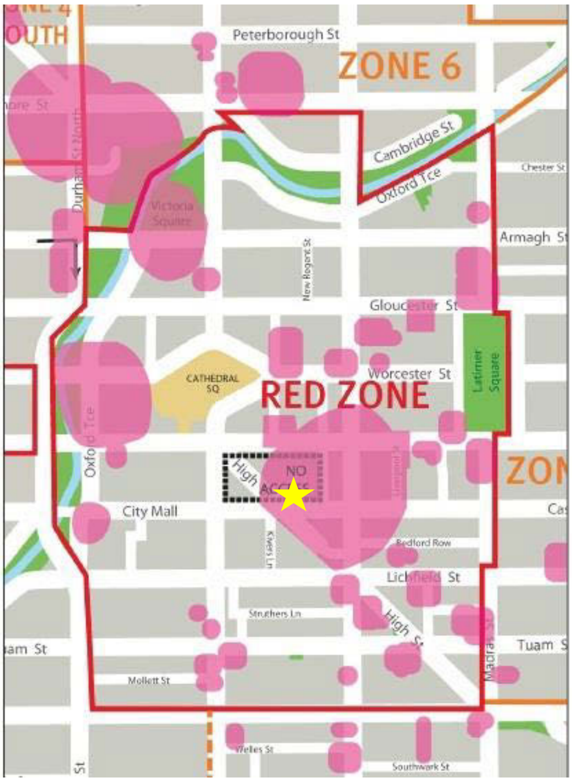
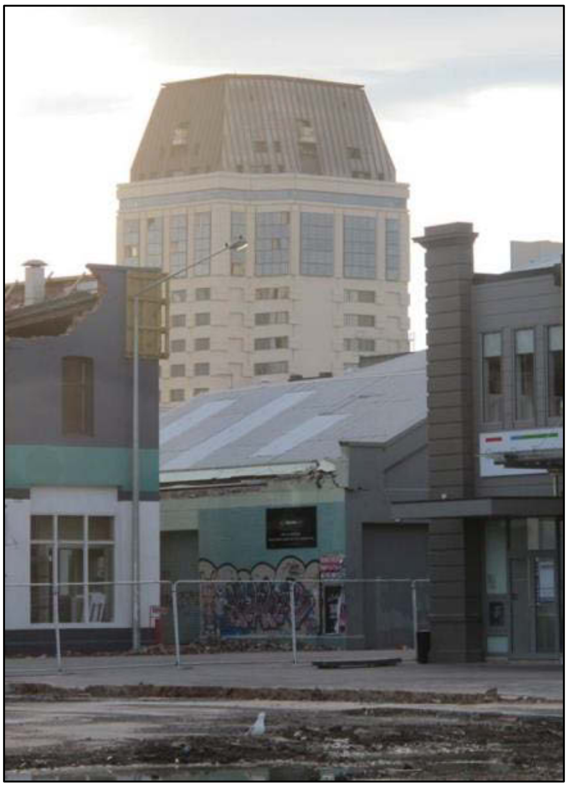


Figure 98 – The Christchurch City cordoned red zone as at 5 March 2011 showing no access areas around dangerous buildings and controlled access green zones. Source: CDEM (2011)

After nearly two weeks this cordoned off area was divided into four Green zones and a Red zone (see Figure 98). Residents and building owners were given controlled access to the Green zones to collect belongings before these zones were reopened to the public. Green zone areas that remained dangerous or red zone areas that no longer needed to be part of the cordon then became Orange Zones. Those still residing within this CBD cordon were required to carry identification and respect a 12 hour curfew at night. Figure 99 shows the location of the Hotel Grand Chancellor that was the source of a major safety concern.



(a)



(b)

Figure 99 – (a)The red zone cordon zone as at 12 April 2011 in the CBD surrounding Cathedral Square (Source: CDEM, 2011). The purple areas denote potential building fall zones (building collapse and debris footprints). The yellow star shows the no access zone around Christchurch's tallest building which was on a visible lean, the 26-storey Hotel Grand Chancellor, shown in (b).

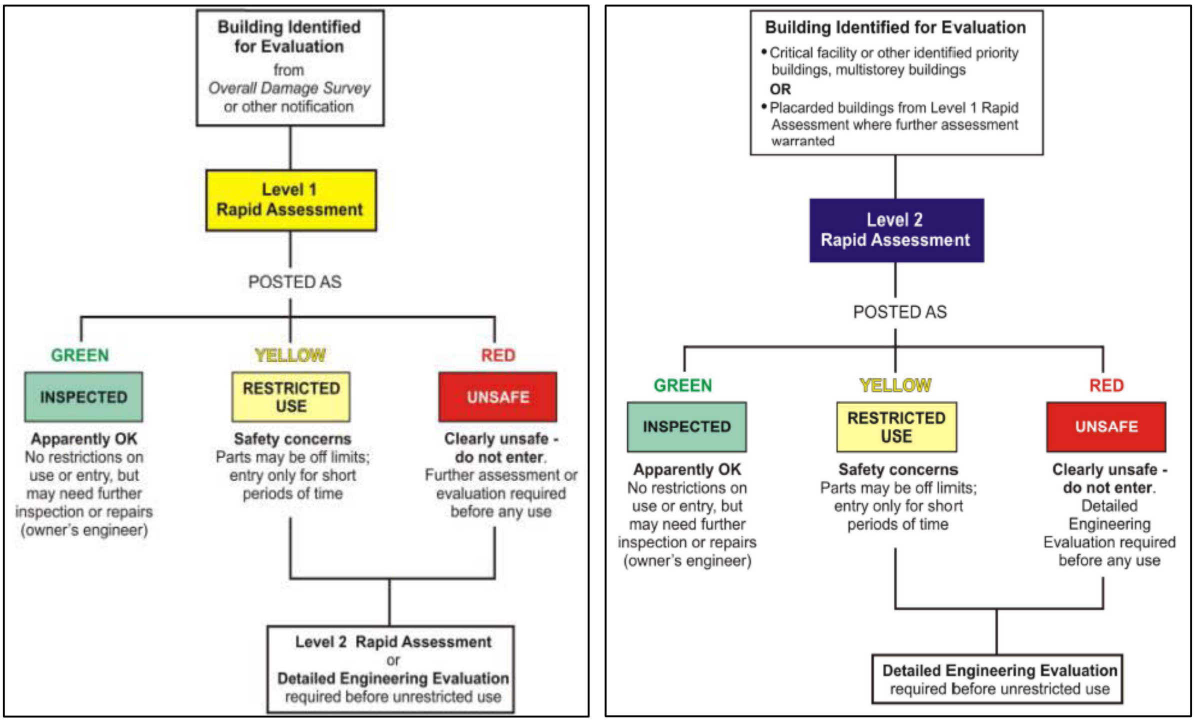
International USAR

Once the Christchurch International Airport reopened for international flights on the 23rd February, 2011, USAR personnel were flown in to assist in rescue, victim recovery, debris clearance, controlled demolition, shoring and stabilisation. Eight international search and rescue task forces joined the three New Zealand teams already engaged in rescue operations. The rescue phase ended on the 3rd March and in total seventy people were rescued alive (CCC, 2011).

Some of the USAR teams trialled an earthquake early warning system which delivered a warning message to mobiles giving teams 3 seconds to get clear of falling debris in the CBD. The system was later used by geotechnical engineers to give warnings of potential rock falls triggered by aftershocks (Wright, 2011).

Building Safety Evaluations

CDEM activated a building safety evaluation process using a triage placard system to indicate whether buildings were safe to enter. The system developed by the New Zealand Society for Earthquake Engineers (NZSEE) is adapted from ATC-20. This is reportedly the fourth time the system has been used by New Zealand following an earthquake (Gisborne 2007; Padang, Indonesia, 2009; and Canterbury 2010) with refinements made to the system after each operation. The aggregated results were used to make decisions on controlling traffic, cordons, safe access corridors, and to indicate the economic impact of the earthquake (NZSEE, 2010).



(a) (b)

Figure 100 – Flow charts showing the (a) Level 1 and (2) Level 2 Rapid Assessment and Posting Process (Adapted from ATC-20). Source: NZSEE (2010).

Two levels of rapid evaluation were undertaken by teams of qualified engineers: Level 1 Rapid Assessment were conducted by external inspection only; while Level 2 Rapid Assessment where a building is over two storeys and required further inspection (but is stable for entry) in the Level 1 assessment (NZSEE, 2010). See Figure 100 for a flow chart for conducting assessments. The building inspection databases were maintained by the Christchurch City Council for reporting and analysis. Experiences from the 4 September earthquake guided the assessment forms, processes and databases.

Several EEFIT team members observed and assisted Building Safety Evaluation teams in the CBD over 3 days during the mission, gaining experience of the triage placard system and undertaking detailed observation of earthquake damage to the interior of many different structures.

At the outset there was some public confusion voiced over the meaning of the placards. CDEM repeatedly reinforced messages that red placards were not a demolition order and that placard colour related only to entry and further inspection. If structural damage was suspected, further detailed engineering assessments should be made. Reasons for red cards included (CDEM, 2011):

- The building was badly damaged or structurally unsound
- The building was in the fall zone of an unstable neighbouring building
- The building was in the path of potential rock falls or land slips

An example of a red placarded building is shown in Figure 101.

In some suburbs black and white sheets were used in place of yellow or green placards (CDEM, 2011). The placard-status of buildings was flexible during the assessment process as many buildings needed to be resurveyed after aftershocks or additional geotechnical surveys. Indicator buildings of varying types of construction were monitored for further damage following aftershocks – further damage could trigger a reassessment of all buildings previously evaluated.

Owners of lightly damaged green placarded buildings were directed to the Earthquake Commission (EQC) whereas owners of yellow or red placarded residential buildings were advised to contact the Christchurch City Council to arrange structural checks by Chartered Professional Structural Engineers. Commercial building owners were advised to directly contact Chartered Professional Structural Engineers for detailed structural assessments.

(a)

(b)

Figure 101 – (a) Red placard on an addition to the Library Chambers built in 1923 (highlighted with a yellow star); (b) cracks in the masonry and parapets secured with strapping on the Library Chambers. Built in 1875, the chambers housed the Canterbury Public Library for over 100 years.

As at 30 March 2011 CDEM reported the building inspection statistics shown in Table 8

(CDEM, 2011).

Table 8 – Building inspection statistics at 30 March 2011 (CDEM, 2011).

		Inspected	Red	Yellow	Green
Outside CBD	Residential	61,496	1824	-	-
	Commercial	5185	964	1107	3114
	Heritage	1150	414		
Within CBD	Commercial & Residential	4354	1069	1022	2263
	Total	72185	4271	2129	5377

Finalised and disaggregated numbers are yet to be published. The numbers in Table 8 are the most recent indication of progress available at the time of this report. The delay in reporting the final statistics relates to secondary hazard analysis to assess whether buildings and land are still viable after geotechnical hazards have been stabilised and flood hazard risks thoroughly assessed.

As noted in Section 7, the risk from unstable cliffs and boulders has caused the evacuation of hundreds of residents in suburbs including Sumner, Redcliffs, Cashmere and Lyttelton. In some cases residents were evacuated from newly built properties which sustained little earthquake damage, but were located in the path of unstable cliffs and rocks. Detailed geotechnical inspections were undertaken by teams of engineers to identify source and hazard zones. At the same time scientists worked to improve rock fall hazard models specific to the volcanic rock of the Port Hills. At the time of writing this report, rock scaling operations were being undertaken to stabilise cliffs and rocks.

Deconstruction and demolition

As part of USAR operations a number of buildings were deconstructed for rescue or recovery purposes. More extensive demolitions of dangerous buildings by contractors soon followed. Some of the early demolition work within the red zone prompted complaints and protests from local business owners who wished to supervise the demolitions or retrieve items to allow them to restart businesses in new locations. Tensions over demolition work and access to the red zone culminated in a three day demolition moratorium, followed by a new mechanism for controlled access: From the 1 April 2011, Chartered Professional Structural Engineers could gain access to the CBD to carry out structural safety assessments and review placarded buildings (CDEM, 2011). Once the engineers submitted their reports to Civil Defence, building and business owners could make an application for controlled access to cordoned areas supervised by engineers and USAR personnel.

As at the 27 April, the current list of buildings to be demolished was 241 (86 heritage) with 82 (22 heritage) to be partially demolished and 39 (22 heritage) to make safe (CDEM, 2011). An example of a heritage building on the demolition list can be seen in Figure 102

Figure 102 – One of Christchurch's most notable heritage buildings, the Cathedral of the Blessed Sacrament, built in 1905. The cathedral was closed for repairs and seismic strengthening after the 4 September earthquake. The 22 February earthquake put the building on the demolition list.

Transition to Recovery: Canterbury Earthquake Recovery Authority (CERA)

At the end of March 2011, the Government announced the formation of a new government department to lead the recovery called the Canterbury Earthquake Recovery Authority (CERA). The Canterbury Earthquake Recovery Bill 2011 was passed under urgency through parliament and sets out the particular functions of the Minister of Earthquake Recovery and the chief executive of CERA including preparation of a Recovery Strategy for Greater Christchurch. CERA was to work with local councils and communities, the residents of greater Christchurch, Ngai Tahu (local 'iwi' – Maori for 'tribe'), the non-government sector and business interests (NZ Legislation, 2011).

The challenge facing CERA will be balancing these interests while leading the recovery process at an acceptable pace. Concerns have been voiced over the possibility of a top-down fast-track approach without adequate community-led public consultation and participation (The Press, 2011c). One of the elements of CERA's structure will be a forum of Canterbury community leaders to ensure CERA reflects issues important to local people (NZ Legislation, 2011). The Council and the Government have announced that they will work together on the Recovery Strategy and are expecting a finalised plan for the CBD within nine months. In that time, the Earthquake Commission and the insurance companies will be completing their work with the property owners.

Specific powers set out under the Canterbury Earthquake Recovery Act 2011 enable CERA to obtain information from any source (including commission of reports), to requisition and build on land, and to carry out demolitions. This wide sweeping power encapsulates many unchartered challenges for New Zealand. CERA will require robust information management policies to deal with the forthcoming hurdles of coordinating and handling sensitive commercial information.

Emergency Information Management

Up to 30th April 2011 when responsibilities transferred to CERA, the exchange of information was coordinated by the Ministry of Civil Defence & Emergency Management (MCDEM) working closely with the Christchurch City Council, Environment Canterbury and many other key organisations. Emergency Information Management (IM) policies reflect the accelerated need for information in compressed time. New Zealand has good baseline datasets and science accessible in emergencies through collaborative initiatives such as GeoNet and the Natural Hazard Research Platform supported by organisations such as GNS Science, EQC and LINZ.

During the emergency, experiences from 4th September and the scale of information exchange required from the 22nd February fuelled some demanding policies concerning collaboration, transparency and information exchange. The view from the emergency management side was that all information from the emergency should be shared unless there was good reason otherwise such as serious security, safety or commercial sensitivity concerns.

During the building and geotechnical inspections, engineers from private firms, EQC and government compiled lists of who had what data and at which levels it could be accessed - this knowledge of its existence was enough to streamline IM processes. A collaborative online centralised repository of borehole and site investigation data is just one improvement achieved since the 4th September earthquake.

This event has exemplified the use of social media for both official and unofficial communication of information. In the early stages of the emergency, when local official capacities temporarily dipped below functional within the city, volunteers established online systems to bridge the gap in public information. Various channels of information sharing include:

- Ministry of Civil Defence & Emergency Management (MCDEM) Public Information Management (PIM) office and key spokespersons
- Bulk message targeted SMS service: early communications and later Earthquake Recovery News and Information (ERNI) service to help inner city residents stay up to date with the almost daily changes affecting them in the red zone
- Christchurch City Council and Environment Canterbury public information site at http://canterburyearthquake.org.nz/
- Twitter (http://twitter.com/#!/NZcivildefence) updates including information on closure/availability of key infrastructure and areas, essential contact numbers, advice in case of aftershocks, progress on opening CBD cordons and updates on structural assessment, places to volunteer or to receive help
- Earthquake Incident Viewer (www.eqviewer.org) provided up to date information to Christchurch residents on operations and services including status of transportation, location of shelters, water and sanitation, and medical centres.

- Multiple instances of emergency crowd-sourcing applications were amalgamated into one Ushahidi powered application, the Christchurch Recovery Map (www.eq.org.nz) (see Figure 103). The site, run by volunteer groups, provided two way communication where the public could exchange any local information they might have that could help others or show they needed help themselves (Leson, 2011). Messages from the public were filtered and displayed in categories to ensure relevance and simplify the huge volume of publically-contributed information. After one month the site ceased updating once public updates decreased and normal communication channels were functioning.

Online initiatives for matching residents needs with available resources included:

- Google person finder – matching those looking for someone to those who have information on someone: http://christchurch-2011.person-finder.appspot.com/
- Trade Me – finding and offering temporary accommodation
- Department of Internal Affairs – assisting with IT and communications

GIS teams from the EOC compiled data from their service providers into corporate online web based GIS applications accessible to those working on the emergency. The applications were used for tasks including mapping damage, monitoring service requests against distribution of resources to identify gaps, monitoring the progress of repairs across the city, and monitoring the building safety evaluation process.

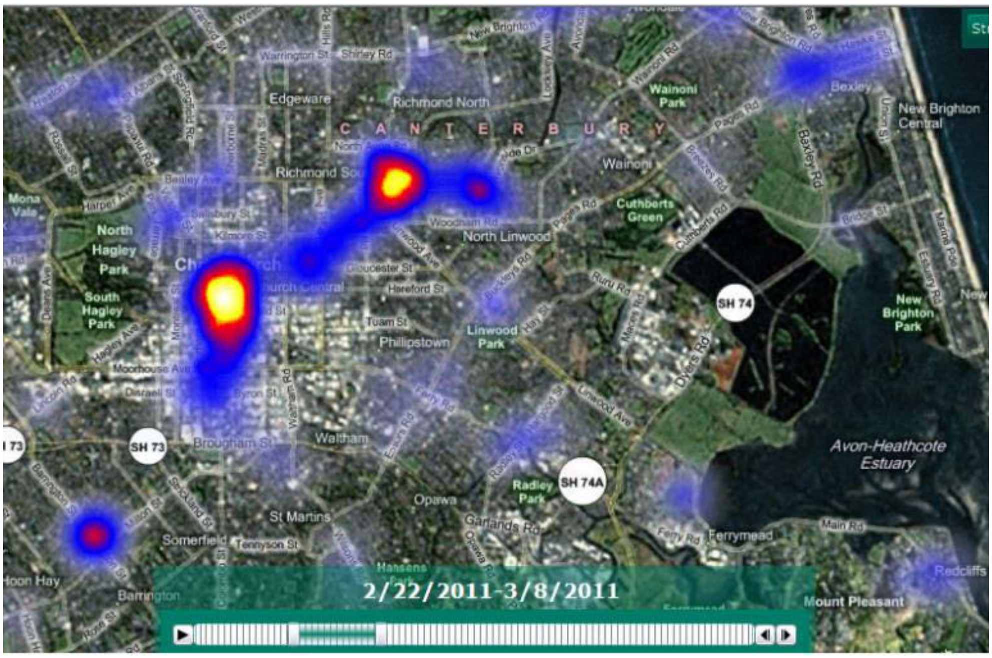


Figure 103 – Ushahidi trend information filtered by 'hazard' category between the dates of 22nd February to 8th March. The yellow and red hot spots indicate high activity (Source: www.eqviewer.co.nz)

Use of Remote Sensing in the Emergency

Remotely sensed data was widely leveraged as part of the emergency IM strategy providing a window in to the situation before a complete picture on the ground could be established. The International Charter on Space and Major Disasters was activated to provide satellite based damage assessments.

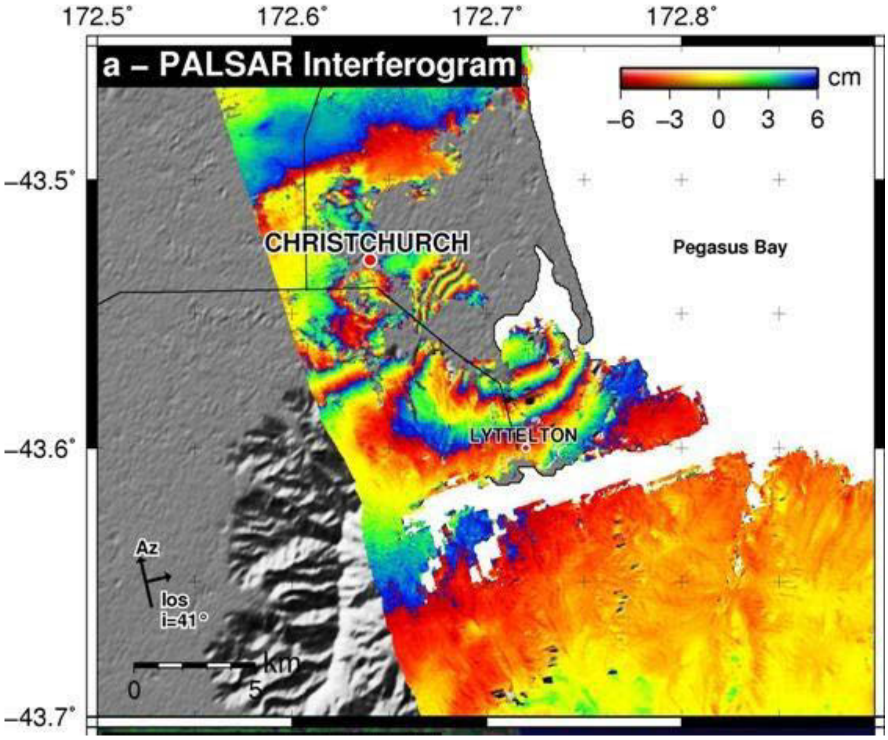


Figure 104 – InSAR showing that the peak ground motion was almost 50 cm of motion towards the satellite (Source: *Elliott, Zhenhong, Parsons, 2011)*

InSAR images from the Alos Palsar were also one of the first products available and provided valuable information about the source fault, see Figure 104.

Within days of the earthquake New Zealand Aerial Mapping (NZAM) acquired 10 cm aerial photography and LIDAR over the Christchurch City and Lyttelton areas. The aerial photography was used to help coordinate USAR operations. Once victim recovery was complete, aerial photography was released to the public under a creative commons license, enabling residents and business owners to interactively view the damage inside the cordon areas for the first time (LINZ, 2011). Due to continued risk of building collapse and falling debris, the aerial imagery provided the public a means to understand the extent of the severe damage in areas of restricted access.

The LIDAR took more time to process, but was integral to the response operations for tasks that included assessing changes in elevation, identifying geotechnical hazards, assessing flood hazards, quantifying debris volume and measuring the movement of buildings. NZAM datasets were also available from the 4 September 2010, earthquake and an earlier survey in 2003 providing a useful baseline for detecting changes in elevation.

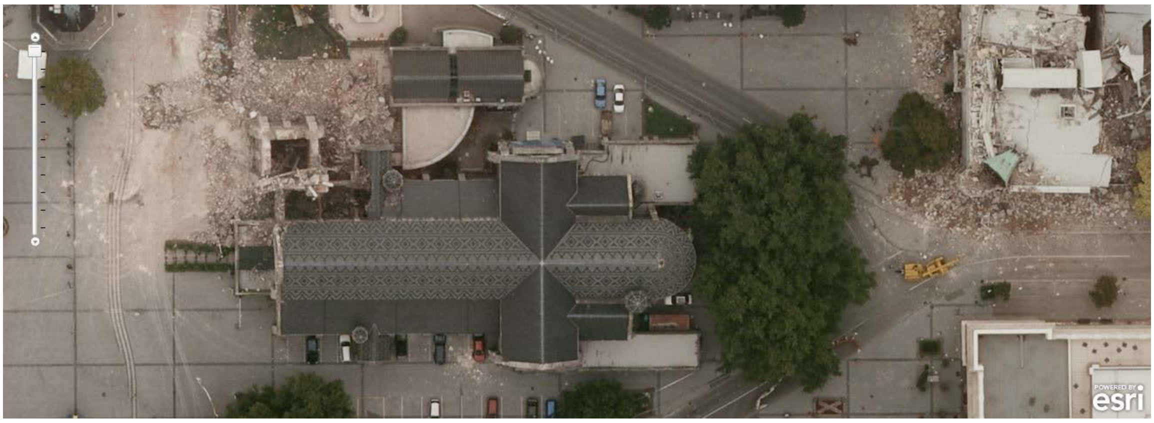


Figure 105 – Damage and debris around the Christchurch Cathedral (Source: NZAM, 2011 Creative Commons Attribution 3.0 NZ)

Arguably one of the notable things to be learnt from the LIDAR is that parts of the city moved in elevation by hundreds of millimetres. Analysis of LIDAR data will be used in assessments of whether land is viable for rebuilding or not (post-earthquake land elevation now poses a new flood risk in some areas of the city).

9. ECONOMIC LOSSES AND INSURANCE

The economic impacts of this earthquake were significant at the national level, in addition to the local and regional level. The Treasury of the New Zealand government estimates that GDP growth will be around 1.5% lower in 2011 as a result of the Christchurch earthquake alone. This negative impact is, however, likely to be mitigated in 2012 with a boost to the economy through recovery activities – increased investment in infrastructure and rebuilding of residential and commercial sectors in Christchurch.

A large proportion of residential losses from this and earlier events qualify for insurance cover from the Earthquake Commission (EQC), which was established by the New Zealand government in 1945. EQC covers earthquake and war damage for people who purchased fire insurance and is paid out using EQC's Natural Disaster Fund, which is accumulated through collection of insurance premiums. EQC purchases its own international reinsurance cover to provide reimbursement in the event of a large loss, and also has a Government Guarantee that acts to ensure that EQC can always pay out to its cover holders, regardless of the total loss amount.

EQC pays out only on residential property damage in New Zealand, but covers damage sustained as a result of earthquake, natural landslip, volcanic eruption, hydrothermal activity, tsunami (and storm or flood for damage to residential land), and fire following any of these events. Cover is provided for dwellings to a maximum of $NZ 100,000 plus GST (Goods and Service Tax) and for contents to a maximum of $NZ 20,000 plus GST. EQC pays out the lower of either replacement cost or repair cost for the property. Additional cover above the EQC limits could be purchased by the property owners from individual insurance companies if required.

In December 2010, EQC estimated the cost of claims for the 4th September 2010 earthquake to be between $NZ 2.75bn and $NZ 3.5bn, with EQC liable for only the first $NZ 1.5bn (reinsurance cover provides EQC with $NZ 2.5bn of cover above $NZ 1.5bn); leaving $NZ 4.5bn in the Natural Disaster Fund for further events. It was estimated that the final cost of that event would be known by March 2011 when all claims assessments were due to have been completed.

Since 4th September 2010, six significant aftershocks have been designated as individual events for insurance purposes. The claims numbers are presented below. As at 22nd February 2011, EQC had paid a total of $NZ 756.35mn for the six events listed.

As at 21st March 2011, EQC had received over 77,500 claims from the 22nd February event alone; the total number of EQC claims from this event is expected to rise to 120,000 bringing the total number of claims since 4th September 2010 to over 300,000.

AIR Worldwide modelled estimates of total insured losses from the 22nd February 2011 event (residential, commercial and industrial building and contents damages) were in the range $NZ 5bn–$NZ 11.5bn (http://alert.air-worldwide.com), while insurance industry estimates on 18th March stood at between $NZ 8bn and $NZ 16bn (http://www.odt.co.nz). Several major international reinsurers have published their estimated losses from this event, with the most significant being Swiss Re: $NZ 1090mn; Munich Re: $NZ 1360mn (the New Zealand earthquakes combined); Hannover Re: $NZ 285mn; Renaissance Re: $NZ 259mn; Everest Re: $NZ 190mn–286mn.

At the time of writing, there remains significant uncertainty in these figures while insurance claims are still being lodged and assessed, particularly in the CBD where access to buildings remains restricted. As a result of on-going damage assessments and difficulty defining which event caused the loss, it is likely to be some months until a clear picture of the final total loss for each event is established.

Table 9 – EQC residential claims totals / estimates for the Canterbury earthquakes up to and including 22nd February event. Source: EQC Information Advertisement (22 Feb 2011); EQC Media statement (20 March 2011), http://canterbury.eqc.govt.nz/ (21 March 2011). Monetary values in New Zealand Dollars ($NZ).

Event	Claims	Cost to insurance (est.)	Cost to EQC	Claims status
04-Sep-10	156,935	$NZ 2.75bn – $NZ 3.5bn	$NZ 1.5bn	Total (deadline passed)
19-Oct-10	3,176	Tba	Tba	Total (deadline passed)
14-Nov-10	2,139	Tba	Tba	Total (deadline passed)
26-Dec-10	18,193	Tba	Tba	Estimate – subject to change
20-Jan-11	2,829	Tba	Tba	Estimate – subject to change
04-Feb-11	419	Tba	Tba	Estimate – subject to change
22-Feb-11	77,515	$NZ 8bn – $NZ 16bn	Tba	Estimate – subject to change

10. CONCLUSIONS

Although nearly all modern engineered structures survived the earthquake with relatively little structural damage, there are many lessons that can be learnt from the Christchurch earthquake. The major findings of the EEFIT mission are listed here:

A significant number of unreinforced masonry structures, surveyed by the authors as well as by other reconnaissance teams (Weng, Y. K., 2011; Dizhur et al., 2011), behaved poorly and suffered major damage or even collapse. While this is no surprise, it does beg the question of how retrofit programmes should be implemented and enforced. Is the most cost effective strategy to simply replace old masonry structures after an earthquake and if so, what does this say about the importance we confer on life-safety and historical significance? This question is made all the more difficult considering that even the performance of many of the strengthened structures that were surveyed by the team did not perform satisfactorily. Systems like steel and concrete frames did prevent the collapse of horizontal structures, thus saving human lives, yet they did not avoid out-of-plane failure of masonry panels, which constitute a serious hazard to people in the proximity of buildings. Out-of-plane failures also meant that large portions of the original materials and finishes were lost.

As the vast majority of historic structures in Christchurch are beyond economic repair, all those that are repaired will be done so due to their historical significance. Those that are deemed to be worthy of restoration should therefore be used to inform future retrofit policy.

Of the modern structures that suffered significant structural damage, most of this could be attributed to irregularities in the structural system. Whereas the poor seismic performance of irregular structures is well documented, this earthquake demonstrates that a combination of the objectives of the owners and architects and their relationships with structural engineers still result in irregular structures that sometimes do not perform as envisaged. If we accept as inevitable that some degree of irregularity will occur in structures in seismic zones, then it may be that we need either more sophisticated analysis techniques, or, for earthquake design codes to apply greater penalties for irregular structures.

Damage to RC shear walls was observed on a number of occasions. It is believed than some of this damage could again be attributed to irregularities in the structural system.

Whereas many precast concrete details performed adequately, there were a number of notable failures. The seismic adequacy of precast connections should therefore be reassessed.

The non-structural damage resulting from the earthquake was often significant, even in modern structures, and some of the plastic hinging in the structural systems may have rendered them beyond economically viable repair.

Liquefaction was widespread and this caused a great deal of disruption after the event. Damage to buried services due to this liquefaction was also widespread and many were still severed at the time of survey. Foundations also suffered due to liquefaction and this was the cause of much of the structural damage seen in Christchurch. This leaves the city of Christchurch with a difficult decision in terms of the resettling of those areas that are now known to be liquefiable.

Many bridges over water suffered acute rotation of abutments due to lateral spreading and while no bridge collapsed and, at the time of survey, all those visited were back in service, the speed reductions that had to be placed on them were very disruptive. Details that can prevent this phenomenon should be investigated and a hazard assessment made on all bridges on liquefiable soils in seismic regions.

In the aftermath of the earthquake there has been a large number modern buildings that have been deemed to be beyond economic repair and the repair times of those buildings that can be repaired have resulted in severe economic and social disruption. Considering the cost of this earthquake and the disruption that it has caused, this calls into question the current performance targets of only ensuring life-safety for the ultimate limit state earthquake and therefore the performance targets of extensive or irreparable damage should be revisited.

Finally, during the course of the five days spent in Christchurch, the team were impressed by the organisation and resilience of the people of Christchurch, the volunteers from all over New Zealand as well as the Christchurch City Council, Environment Canterbury and the New Zealand Government. In our opinion, the disaster management practice of New Zealand is to be commended.

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http://www.odt.co.nz/news/business/152475/re-insurance-losses-christchurch-rise

http://alert.air-worldwide.com/EventSummary.aspx?e=548&tp=62&c=1

http://canterbury.eqc.govt.nz/publications/info-advert-220211

http://canterbury.eqc.govt.nz/

http://www.eqc.govt.nz/abouteqc.aspx

http://www.eqc.govt.nz/abouteqc/publications/mediastatements/mediastatements/progress-march20.aspx

http://www.naturalhazards.org.nz/

http://www.geonet.org.nz/canterbury-quakes/

http://www.gns.cri.nz/Home/Our-Science/Natural-Hazards/Recent-Events/Canterbury-quake/Recent-aftershock-map

http://www.gns.cri.nz/

APPENDIX A: DETAILED RESIDENTIAL DAMAGE SURVEY

Earthquake damage survey data is important for calibrating fragility and vulnerability relationships for estimation of damage and losses in future events. A difficulty that is often faced in the derivation of these relationships is that the ground shaking intensity can only be estimated. In fact, this is the reason that often macroseismic intensity measures have been used, rather than instrumental measures. This can potentially increase the reliability of the fragility data, but moves the problem to the estimation of intensity indices for a given earthquake scenario. When using instrumental intensity measures, often a ground motion prediction equation is employed to estimate a median level of ground shaking for the study area, which introduces a very large uncertainty to the fragility relationship that is not usually considered.

For this reason, the collection of damage survey data as close as possible to accelerometers, where the level of ground shaking is known with most precision, is of significant importance. Christchurch presents an interesting case study, both for the wide coverage of accelerometers over the city, but also because the moderate magnitude event at close distance produced a relatively large variation in ground motion intensity over the city. This could potentially allow fragility relationships to be derived for a range of instrumental intensity levels.

The EEFIT team carried out a damage survey of residential buildings in three locations close to accelerometers. A fixed distance from the accelerometer was not used – in contrast to other teams who have proposed a 300-foot (90-metre) radius around accelerometers. Survey areas were selected to accommodate a range of peak ground accelerations, and in residential areas to allow a relatively homogeneous building stock to be assessed.

There were several difficulties encountered by the team which compromise the value of the data obtained. Therefore, the results presented in this section should be considered indicative only, and are perhaps of limited use for reliable derivation of fragility curves. The following problems were identified:

- Chimney damage / collapse identified from new or temporary covering on roofs can be incorrectly assigned to the 22nd February event for the following reasons:
    - Ground shaking in the Darfield earthquake of 4th September 2010 and subsequent aftershocks prior to 22nd February caused a significant amount of chimney damage. Anecdotal evidence from several residents around Cashmere High School confirms chimney collapse or damage (leading to removal of chimney stacks) prior to 22nd February.
    - Christchurch has an historical air pollution problem and since 2003 Environment Canterbury has applied rules to reduce the use of open fires, while encouraging the use of alternative heat sources such as heat pumps or

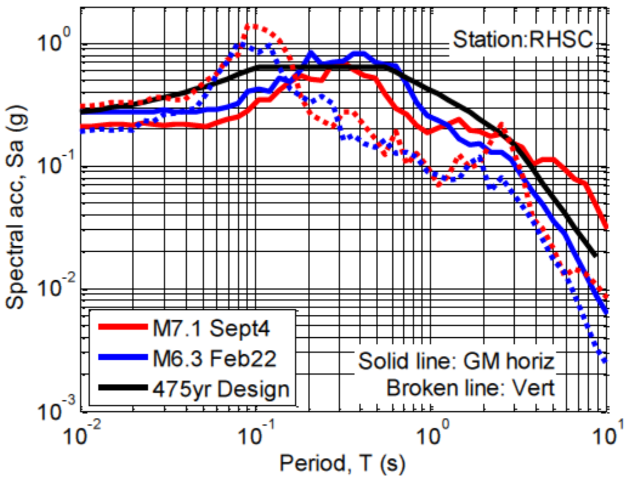
gas heaters (www.cleanheat.co.nz). The effect of the Clean Heat project is that many Christchurch residents do not use their chimney, and may have removed their masonry chimney for reasons entirely unrelated to earthquake occurrence. Where this is the case, patching of roof materials could be misconstrued in surveys as chimney removal due to earthquake damage.

- Some earthquake damage may not be visible from the road, resulting in potentially incorrect classification of building damage. For example, a two storey structure in the Cashmere High School area appeared from the road to be undamaged, however, information from a neighbour suggested that the masonry wall at the rear of the property had required a significant amount of bracing due to earthquake damage.
- Existing damage scales (such as the European Macroseismic Scale) are not well-suited to the building types and degrees of damage typical of the buildings observed. A popular measure of damage based on tagging statistics – red, yellow and green tags relating to safety of entering the building (see Section 8) – is inadequate for estimating monetary losses in the buildings observed. Therefore, the classifications in the following are not based on a common damage scale, but are just for information about the specific types of damage observed.
- Heterogeneity of building stock. As an obvious example, the primary factor in whether a building experienced chimney failure was whether it had a chimney, and, if so, whether it was made of brick. It is difficult to classify all features of a building that could possibly lead to damage, and, in any case, chimney failure is unlikely to be of most interest for loss estimation studies. It is also the nature of fragility curves based on empirical data that they reflect the overall statistics of buildings in the study area, and they should only be used in loss estimation for other areas if building stock is comparable.

Riccarton High School Recording Station (RHSC)

The first survey area was near the Riccarton High School recording station (Figure 106a), where ground shaking intensity was relatively low (Figure 106b). From the spectrum in Figure 106(b), the horizontal shaking from the February event was comparable to the design spectrum for Christchurch, and was stronger than the September event for periods less than 1.0 seconds (of most relevance for residential buildings).

(a)



(b)

Figure 106 – Riccarton High School accelerometer (a) location of station (yellow circle), 90-m radius around station (red circle) and survey area (yellow area) (marked up Google Maps image);(b) response spectra (Bradley, 2011).

Very little damage was observed in this area (Figure 107), with the exception of some damage to chimneys or roof tiles. As noted above, it was difficult to attribute this damage specifically to the February earthquake.

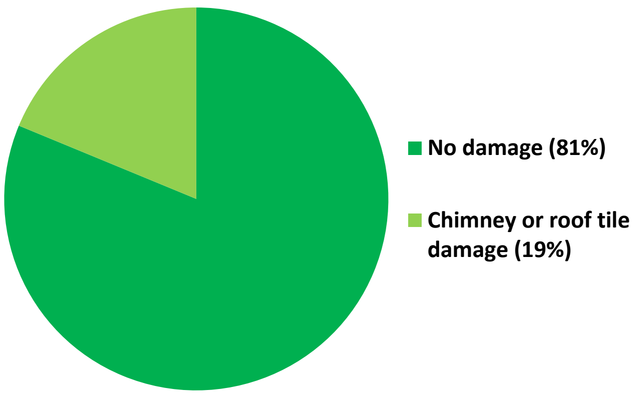
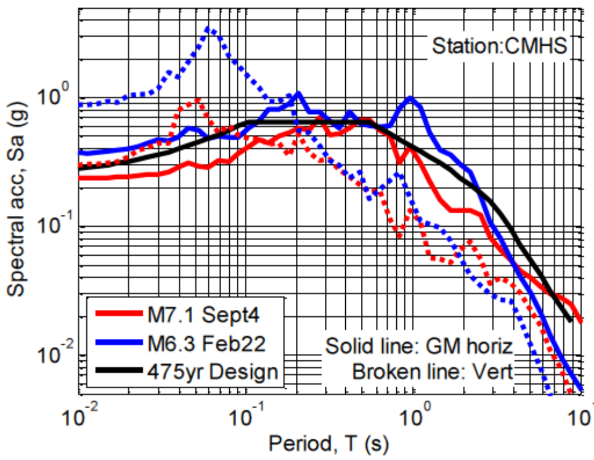


Figure 107 – Observed damage statistics for residential buildings in Riccarton High School study area.

Christchurch Cashmere High School Recording Station (CCHS)

The second survey area, in the area surrounding Christchurch Cashmere High School recording station (Figure 108a), experienced slightly higher levels of ground shaking than that in the previous section, and significantly stronger vertical accelerations (around 0.9g peak) (Figure 108b).

(a)



(b)

Figure 108 – Christchurch Cashmere High School accelerometer (a) location of station (yellow circle), 90-m radius around station (red circle) and survey area (yellow area) (marked up Google Maps image);(b) response spectra (Bradley, 2011).

Since the area also comprised the riverside property along Ashgrove Terrace, the observed damage also included significant areas of liquefaction and liquefaction-induced damage (Figure 109). Aside from the higher levels of ground shaking in this area (compared with the previous one), this helps to explain the slightly higher proportion of damaged buildings, including around 9% with foundation problems.

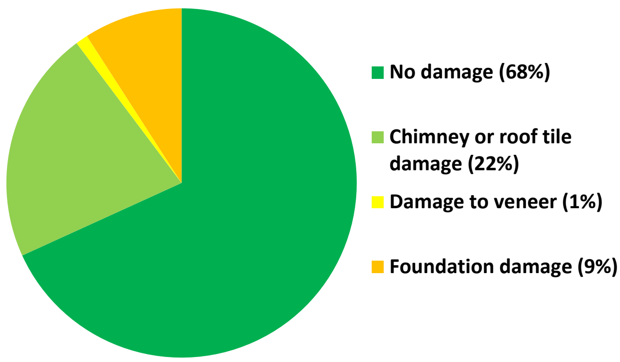
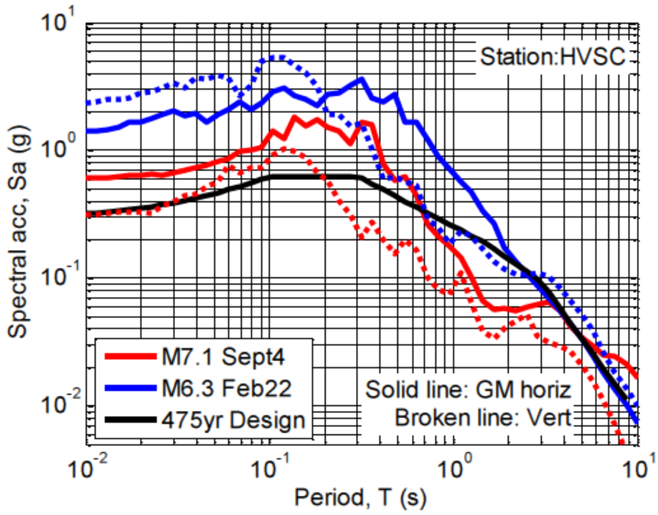


Figure 109 – Observed damage statistics for residential buildings in Christchurch Cashmere High School study area.

HEATHCOTE VALLEY SCHOOL RECORDING STATION (HVSC)

The Heathcote Valley School recording station (Figure 110a) recorded the largest horizontal and vertical accelerations in the earthquake, due to its proximity to the epicentre (Figure 110b).

(a)



(b)

Figure 110 – Heathcote Valley School accelerometer (a) location of station (yellow circle), 90-m radius around station (red circle) and survey area (yellow area) (marked up Google Maps image);(b) response spectra (Bradley, 2011).

Observed building damage was much more severe, as could be expected for peak ground accelerations of 1.4g and 2.1g (horizontal and vertical). There were several examples of buildings that had moved up to 300 mm off their foundations, which could be expected with vertical ground motions in excess of gravity. There were also examples of partial and full collapse of residential buildings.

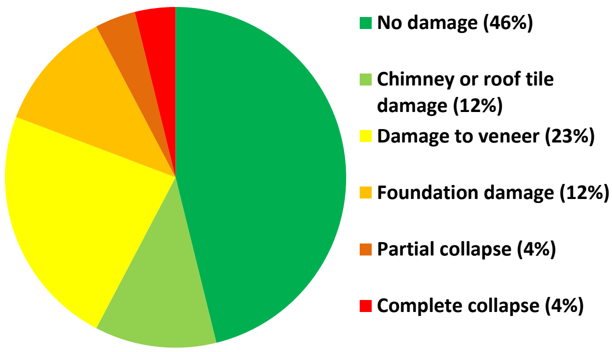


Figure 111 – Observed damage statistics for residential buildings in Heathcote Valley School study area.

EEFIT

Earthquake Engineering Field Investigation Team

EEFIT is a UK based group of earthquake engineers, architects and scientists who seek to collaborate with colleagues in earthquake prone countries in the task of improving the seismic resistance of both traditional and engineered structures. It was formed in 1982 as a joint venture between universities and industry, it has the support of the Institution of Structural Engineers and of the Institution of Civil Engineers through its associated society SECED (the British national section of the International Association for Earthquake Engineering).

EEFIT exists to facilitate the formation of investigation teams which are able to undertake, at short notice, field studies following major damaging earthquakes. The main objectives are to collect data and make observations leading to improvements in design methods and techniques for strengthening and retrofit, and where appropriate to initiate longer term studies. EEFIT also provides an opportunity for field training for engineers who are involved with earthquake-resistant design in practice and research.

EEFIT is an unincorporated association with a constitution and an elected management committee that is responsible for running it activities. EEFIT is financed solely by membership subscriptions from its individual members and corporate members. Its secretariat is generously provided by the Institution of Structural Engineers and this long-standing relationship means that EEFIT is now considered part of the Institution.

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