OBSERVATIONS AND IMPLICATIONS OF DAMAGE FROM THE MAGNITUDE M_w 6.3 CHRISTCHURCH, NEW ZEALAND EARTHQUAKE OF 22ND FEBRUARY 2011

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ABSTRACT

This paper describes the observations made by a reconnaissance team following the 22nd February 2011, M_w 6.3, Christchurch, New Zealand earthquake (GNS Science; 2011). The team comprised of members of the UK based Earthquake Engineering Field Investigation Team (EEFIT) who spent five days collecting observations on damage resulting from the earthquake. Although the magnitude of this earthquake was not particularly high (M_w 6.3), the shallow focus and close proximity resulted in locally very high ground motions, widespread damage and 182 fatalities. The earthquake is also particularly notable for the widespread liquefaction it caused, landslides and rockfalls in the hills south of Christchurch, and the significant damage suffered by unreinforced masonry and historic structures. Over wide areas of central Christchurch, recorded accelerations were in excess of those required by the current New Zealand seismic loadings standard (NZS1170.5:2004): Standards New Zealand (2004), and therefore the earthquake presented a valuable opportunity to assess performance of modern buildings under code-level ground acceleration.

INTRODUCTION

At 04:35 (local time) on the 4th September 2010 the province of Canterbury suffered a magnitude M_w 7.1 earthquake. The epicentre of this earthquake was approximately 40 km West of Christchurch near the town of Darfield (GNS Science, 2010). The earthquake was notable for the widespread liquefaction, the large amount of non-structural damage and the damage to unreinforced masonry (URM) structures in the City of Christchurch and its surrounding areas. However, no modern engineered structure collapsed and few suffered significant structural damage and no lives were lost. As is usual with seismic events of this magnitude, the region suffered a series of aftershocks. The most notable of these occurred on the 26th December 2011 (M_w 5.9), on the 22nd February 2011 (M_w 6.3) and, subsequent to the EEFIT reconnaissance, on the 13 th June 2011 (M_w 6.3). The main topic of this paper is the most damaging of these events, the M_w 6.3 earthquake on the 22nd February 2011 which occurred at 12:51 (local time). The close proximity of the epicentre to Christchurch and its shallow focus resulted in widespread structural damage, collapse of buildings, disruption to services and the loss of 182 lives and a further 164 serious injuries. The earthquake was destructive with macroseismic intensity of up to 9 on the EMS98 scale. The economic costs of this earthquake are far reaching. The Treasury of the New Zealand government estimates that GDP growth will be around 1.5% lower in 2011 as a result of this earthquake alone, while many insurers have placed the insured cost in the tens of billions. AIR Worldwide estimates total insured losses from the 22nd February event to be in the range $NZ 5 bn–$NZ 11.5 bn (http://alert.air-worldwide.com), while insurance industry estimates on 18th March stood at between $NZ 8 bn and $NZ 16 bn (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 1.09bn; Munich Re: $NZ 1.36bn.

This report presents the observations and findings of a reconnaissance mission conducted by the UK based Earthquake Engineering Field Investigation team (EEFIT) to the City of Christchurch and the surrounding areas.

BACKGROUND TO THE MISSION

The reconnaissance team consisted of eight members (the authors of this paper) with a wide range of expertise including engineering seismology, engineering geology, geotechnical earthquake engineering, earthquake engineering, structural engineering, lifeline engineering and seismic risk management. The mission lasted for five days and started on the 12th March 2011. EEFIT was also helped by many people in New Zealand, and they are listed in the acknowledgements section.

Whereas the magnitude of this earthquake was not particularly large, the earthquake is significant for a number of reasons. From a seismological point of view it is significant for its high ground motions especially in the vertical direction. From a geotechnical point of view for the extent of liquefaction within an urban area, and from an engineering point of view, this earthquake is important as it occurred in a developed region with not only modern building codes, but what can be considered a state-of-the-practice construction industry, and therefore provides the opportunity to assess modern seismic practice. In particular, many heritage structures were rendered beyond economic repair as were many URM structures. As the New Zealand building stock and design codes are similar to many other developed regions, the lessons that this earthquake can offer have global implications. The relatively low magnitude, close proximity and shallow focus means that this event is also of interest to regions of relatively low seismicity where a magnitude of this size might be considered an extreme event. The final important area of note for this earthquake is that the measured ground accelerations exceeded the design ground accelerations over a wide area of Christchurch and so this event presents the opportunity to assess the life-safety and collapse performance limits of the modern, seismically designed structures in this region.

The main aim of the mission was to identify lessons that could be learnt for seismic design based on observations of failed buildings, infrastructure and geotechnical performance. In particular:

- To assess how well modern structures designed to modern New Zealand seismic design standards performed;
- To identify modern structures which did not perform adequately, and therefore possibly identify deficiencies in the current design standards;
- To observe the extent and severity of liquefaction in the area and its impact on buildings and lifelines;
- To observe the performance of masonry structures - reinforced, unreinforced - and to assess the adequacy of any retrofitted masonry structures;
- To observe and assess the performance of geotechnical structures and foundations;
- To investigate the nature and impact of landslides in the area; and
- To observe the disaster relief activities conducted by the New Zealand Government and other relevant organisations.

REGIONAL SEISMICITY

New Zealand is located on the tectonic plate boundary between the Australian Plate to the west and the Pacific Plate to the east. 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 Puysegur 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.

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 southwest 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 distributed within the Southern Alps and the remaining approximately 5 % on a 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 is the Alpine Fault but there are a number of additional mapped active faults within the Southern Alps (Stirling et al., 2008). Few 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 regarded as 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 (and east of the North Island). More diffuse, shallow seismicity occurs in the central region of the South Island. 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 and the July 2009 Dusky Sound $M_w = 7.8$ event (GeoNet, 2011). The Canterbury Plains are a region of lower seismicity with fewer large earthquakes but numerous small to moderate size earthquake). There is no historical record of a large earthquake in the immediate region of Darfield or Christchurch. However, it should be noted that the written historical record in New Zealand is relatively short at about 170 years. The two nearest historical felt events to Christchurch are a magnitude $M_w = 4.7$ to 4.9 earthquake centred near Christchurch in June 1869 and a magnitude $M_w = 5.6$ to 5.8 earthquake near Lake Ellismere in 1870 (Forsyth et al., 2008; NZ Earthquake Catalogue).

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 and Quaternary (23 million years to present).

The basement rocks are Paleozoic to 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 Late Cretaceous (100 to 65 miilion years); one of these, in the Miocene (23 to 5 million years) formed the Banks Peninsula (Forsyth et al., 2008). The 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 (Forsyth et al., 2008). During more recent geological time, the Quaternary (age), the region has been affected by ongoing active tectonics, glacial processes and fluctuating sea levels. 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 of sand spits and coastal 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.

The near surface geology (upper 20m) of Christchurch is dominated by interdigitation of Springston (alluvial) and Christchurch Formations (marine). In the subsurface the alluvium probably interdigitates with marine silts, sands, clays, sand of the Christchurch Formation and coastal dunes composed of loose sand (Forsyth et al., 2008; Brown and Webber, 1992).

GROUND MOTIONS

The Darfield earthquake, and subsequent aftershocks, were very well recorded by both the national-scale GeoNet network of strong-motion instruments GeoNet (2011) 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 available 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. The highest peak acceleration had a value of 1.26 g, 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 (Cousins and McVerry, 2010).

The measured ground motions from the Christchurch earthquake were generally higher in the near field than those measured during the Darfield earthquake. The highest peak value of 2.2 g was measured at a station (Heathcote Valley Primary School) 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 0.95 g in the Lyttelton area. Peak ground motions throughout downtown Christchurch were generally high in the range of 0.5 to 0.8 g (GeoNet.org.nz). The peak ground acceleration values recorded in central Christchurch were 3 to 6 or more times higher during the Christchurch earthquake than during the Darfield earthquake.

Comparison of the response spectra for the record ground motions with the 475 year return period level spectrum has been made by Bradley (2011) and these indicate 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 (5km).
- 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 higher 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.

New Zealand Building Stock and Design Practice

Current Design Practices in New Zealand

New Zealand has a high standard of seismic design practice, and many modern seismic design principles originated in New Zealand's universities and engineering consultancies. As with other areas of high seismic activity, design standards have evolved in response to earthquakes, both within the country and abroad. To understand how seismic resistance varies with the age of different buildings, it is important to understand the various milestones in seismic engineering practice. These are presented in Table 1 and are based primarily on Davenport (2004) and Beattie et al. (2008).

Current structural design in New Zealand is controlled by the Building Code, 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.

To assess whether buildings met their design objectives during the earthquake it is necessary to understand the specific performance requirement of the seismic code. These are presented in Table

2, which is an excerpt from a larger table in (PCFOG, 2009) and 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). The ground motion intensities used for design according to NZS 1170.5:2004 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 calculations are carried out for a 1/500 (= 0.2%) annual probability of exceedance, and serviceability checks are carried out for a 1/25 (= 4%) annual probability of exceedance.

Table 1 – Summary of development of NZ seismic design practice (after Davenport (2004) and Beattie et al. (2008))

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.
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 2 – 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 1170.5:2004 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 a strong 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.

As in other seismic design standards, the spectral shape for rock is adjusted based on discrete subsoil site classification. NZS 1170.5:2004 specifies a hierarchy of preferred site classification methods. If possible, sites should be classified into one of five categories (A-E) according to site period (a function of both shear wave velocity and depth of soil strata), other methods incluce borelogs with or without quantitative measurements of geotechnical properties, and, finally, by surface geology and estimates of soil depth. In contrast to both Eurocode 8 and ASCE 7-10, ranges of values of $V_{s,30}$ (average shear wave velocity in top 30 m of soil) are only given for the first two site classes ("strong rock" and "rock" – both with identical design spectra). Softer sites ("shallow soil", "deep or soft soil", "very soft soil") are described in terms of site period and the depth of soil strata at the site, which therefore can take into account depths greater than 30 m.

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. 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.

A rigorous correlation between earthquake intensity and damage is beyond the scope of this paper; however, readers are directed to Bradley (2011) where response spectra derived from the ground motions measured at various stations are compared to design spectra. This analysis shows that in many parts of Christchurch the experienced ground accelerations were significantly greater than the design accelerations.

Performance of Buildings

Timber Structures

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 are 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 1.

The damage observed by the survey team was primarily driven by 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 paper. Another typical failure mode for timber structures was due to insufficient tying to foundation, particularly problematic in areas where the vertical acceleration exceeded 1g. A typical example of a residential timber structure that has moved off its foundation is presented in Figure 1a. 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 (Figure 1b). Several instances of soft storey failure in a timber building were also observed (Figure 1c).

Reinforced Concrete Structures

Reinforced concrete (RC) is the most widespread construction material used in the Christchurch Central Business District (CBD) and other commercial areas. The majority of RC buildings performed adequately, in that they satisfied the performance objectives outlined in Table 2. A few notable failures were picked up on by local and international news media, especially the Canterbury Television (CTV) and Pyne Gould Corporation (PGC) buildings, which were responsible for 116 and 18 of the total fatalities, respectively (Chief Coroner (2011)). Most failures could be attributed to irregularity in plan and/or elevation, or inadequate detailing (moment frame beam-column joint reinforcement; wall boundary zones).

For the most part, moment frame buildings performed well, and most damage was caused by differential settlement (see Foundations section). Minor flexural cracking was observed in beams (Figure 2b) and at column bases (Figure 2a) for the most part, 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 3b 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.

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 4. 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 4b). 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 (Hyland Fatigue and Earthquake Engineering, 2012; Beca Carter Hollings and Ferner Ltd, 2011b).

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 5). On the day following the earthquake, rescue teams were evacuated from the area around the building due to the significant lean. This has lean was later established as, the south-east corner of the building dropping by approximately 800mm and deflecting horizontally approximately 1300mm at the top of the building (Dunning Thornton Consultants Ltd: 2011) . The building has subsequently been stabilised for eventual demolition. Due to the unstable nature of the building, access was not possible, but 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 to the car park at the ground level (EERI 2011). The driveway is visible at the bottom right of Figure 5a and the car park can be seen in Figure 5b.

Various degrees of damage were observed to RC structural walls, from no visible cracking, to moderate cracking in plastic hinge zones (Figure 6), to significant crushing of concrete and buckling of longitudinal bars in boundary zones (Figure 7).

The wall shown in Figure 7 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 – 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.

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 storeys of the 17-storey Forsyth Barr building. The stairwell prevented evacuation of the occupants, and impeded search and rescue efforts, clearly not satisfying the "unassisted evacuation possible" objective from Table 2. Figure 8a shows the building, with windows broken and labelled at each level by search and rescue teams, and a crane being used to allow occupants to retrieve possessions from the building. Beca Carter Hollings and Ferner Ltd (2011a) suggest the staircase failure appears to have been caused by inadequate movement allowance in the stair seating detail, causing the stairs to form a compression strut between floors. This was exacerbated by at least some of the movement joints being filled with construction debris, packing material or mortar.

The team also observed damage at the top of stairwells as part of the Level 2 (internal) inspections. Figure 8b shows the exposed seating detail, with a seating angle of 60 mm, and a 15 mm gap (measured after the earthquake). It is not known how much gap/seating was initially provided, but the observed damage suggests that impact did occur during the earthquake.

Precast concrete is also used in New Zealand for beams, columns and structural walls (Park, 2002). Figure 9a 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. Initially, during the team's visit, larger precast structures, including the AMI stadium shown in Figure 9b, also appeared to have performed well with very little apparent external damage. However, the severity of the damage to the stadium became clearer after the visit, and several stands of the stadium are expected to be demolished, demonstrating the difficulty in drawing conclusions based on external assessments only.

In contrast to Europe, flat slab construction is rare, although a few examples were encountered and these 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 10. This structure included 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 survey team's visit.

Steel buildings

There are relatively few steel buildings in Christchurch (and New Zealand generally), 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). 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 11). The lateral system comprises perimeter eccentrically braced frames (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. Local lightweight bracing around an automated parking elevator was also reported to have failed (Bruneau et al., 2011).

As part of detailed Level 2 inspections, the team 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 12a). The building suffered differential settlement due to liquefaction, and there was evidence of this in ground floor columns and ground-bearing slab. There was no evidence of any damage to steel moment frames at higher levels, including on the floor that was being fitted out and therefore had beams and columns fully exposed (albeit covered in fireproofing; see Figure 12b). An area of fireproofing was removed, which showed no signs of buckling in beam flanges; however a more detailed inspection was not conducted as it would have required extensive removal of fireproofing.

Unreinforced Masonry

42 of the fatalities from this event were associated with failure of URM Cooper, Carter and Fenwick (2011). The survey of URM structures mainly focused on the areas of the CBD and Lyttelton, where this type of construction constitutes a large percentage of the building stock, as also pointed out by other authors (Weng, Y. K., 2011; Dizhur et al., 2011). Masonry structures were also observed in various residential areas, although in smaller number since timber frames have far wider application for residential construction. Surveyed buildings can be grouped into four main categories:

1. Two-storey commercial row brickwork buildings (Figure 13a);
2. Large commercial and residential brickwork buildings either detached or built in rows (Figure 13b);
3. Churches (Figure 13c);
4. Other typologies of buildings with either public or industrial use, such as the old University colleges, old stonework mills, etc. (Figure 13d).

These categories fall in the classes D, E, F and G of the New Zealand's URM building stock as classified by Russell and Ingham (2010).

From the observations gathered in the surveyed areas, URM structures appeared to be the most highly damaged by the earthquake. A large proportion had already suffered some level of damage during the Darfield earthquake of the 4th of September 2010 (Dizhur et al., 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. Due to the lack of information regarding the integrity of the structure prior to February earthquake, it was not always possible to determine from on-site survey the evolution of damage as a consequence of the two separate event; however, in the light of the observations collected for buildings with evident or known damage due to the Darfield earthquake, 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 the surveyed URM buildings and influential to the seismic response were identified and are presented in the following in relation to the observed damage mechanisms.

1. Material quality. Churches and public buildings (Figure 13c, Figure 16c and Figure 16d) 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.

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 14).

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 15 a and Figure 15c). 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 15d and Figure 15e) or arch mechanisms (Figure 15f). 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 15g and Figure 15h).

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

Many of the URM buildings surveyed by the authors had undergone upgrade and retrofit (this is in agreement with the on-site observations already collected by Dizhur et al. (2011)). Despite being strengthened, not all the strengthening systems had achieved the expected levels of performance, partly because of the quality of the existing materials, and partly because of shortfalls in the design and detailing of the strengthening. Recurring strengthening systems, with relative advantages and pitfalls, are briefly reviewed in the following.

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 16a), or of the inner timber frame in those buildings that feature outer masonry structure and inner structure with timber columns and portals. Anchors had an end plate, or were embedded in the masonry and fixed either by resin, or by mechanical connection. Irregularly spaced, insufficiently sized and too far apart elements proved ineffective in avoiding the separation of structural elements (Figure 16b and Figure 16c), whilst regular layouts prevented out-of-plane failures (Figure 16d). The poor quality mortar also affected the performance of anchors, determining premature bond failure

in the mortar joints and thus compromising the transmission of horizontal load between structural elements.

2. Steel bracing and steel frames (Figure 17a and Figure 17b). 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 17b and Figure 17c). In fact, a better global performance is likely to be due to better detailing (Figure 17d).

3. Concrete frames presented the same problems as steel frames: despite behaving well during the earthquake, they were not sufficiently connected to prevent out-of-plane damage to the masonry walls.

Reinforced Masonry

The majority of observed reinforced masonry (RM) structures were two-storey commercial buildings, which performed well and did not suffer any major structural damage (Figure 18). 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 19), and internal blockwork walls.

A notable example of a heavily damaged commercial RM building is shown in Figure 20. The building was on an obtuse-angled street corner, and therefore the layout was almost triangular, one very open façade (Figure 20a) 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 20b). The neighbouring building was also very close (but unconnected) and pounding damage was observed in both buildings (Figure 20c).

Performance of Lifelines

Lifelines were particularly badly affected due to severe and widespread liquefaction (as described further in Geotechnical Observations and Liquefaction section). Settlements of hundreds of millimetres were not unusual and lateral spreading of up to a metre was also often encountered. The liquefaction and lateral spreading had a significant effect on buried services as liquefaction induced settlements, tension cracking due to lateral spreading and floatation of services (Figure 21), 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.

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 resulting from the liquefaction that had been experienced. A number of transformers had suffered settlements due to liquefaction (Figure 22); however it could not be established if this had resulted in disruption of supply. Stevenson et al. report that 60% of households had electricity one day after the earthquake, while 99% of households had electricity one month after the event. These figures do not include the services within the cordon, most of which were out for the entire month after the earthquake. In terms of damage to buried electricity infrastructure, Gionvinazzi et al. (2011) report that 50% of the 66kV underground cable network in Christchurch was damaged.

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 were closed for a few days immediately after the earthquake 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 23a) and a few roads were impassable due to lateral spreading (Figure 23b, Figure 24). 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. In the Eastern suburbs, 83 sections of 57 roads were closed Gionvinazzi et al. (2011).

Bridges

Damage to bridges due to ground shaking was minimal, with no bridges collapsing or significantly moving off their substructures due to ground shaking and damage was generally limited to spalling of concrete resulting from pounding (see Figure 25b), 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 (Figure 25a). Flow of the soil towards the river generated very large lateral forces. These forces pushed the abutment towards the river; however, the bridge decks prevented movement at the top of the abutment (resulting in rotation of the abutment and a corresponding displacement, towards the river, at the base of the abutment). As the base of the abutment was connected to the piles, the tops of these piles, which had insufficient stiffness to resist the resulting lateral forces, moved relative to the bottom of the piles causing a relative rotation between the abutment and the pile and resulting in the observed plastic hinges. It was not possible to determine how far the piles may have moved towards the river however all the bridges inspected had plastic hinges at the tops of the piles and so they must have provided some lateral resistance and it is recommended that this be investigated. The lateral spreading also resulted in settlement of the approach road. The mechanism for this is that the abutments, which are on piles, do not settle vertically; however as the soil spreads towards the river there is 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.

Telecommunication

As far as the team could ascertain, by interviewing locals, at the time of survey much of the telecommunications visited were operational; however, many telegraph poles were leaning significantly and switching boxes had suffered settlements (Figure 26).

Geotechnical Observations and Liquefaction

Liquefaction was a major feature of this earthquake and affected many engineered structures, geotechnical structures and lifelines. As mentioned above, lifelines (transport corridors and buried services) were particularly badly affected, but also many structures experienced severe damage to their foundations as a result of ground deformation. Due to time constraints a comprehensive survey of liquefaction was not possible; however, it can be said that some of the worst cases and greatest extent of liquefaction occurred near the Avon River. This distribution is due to the predominance of loose, saturated alluvial soils in these areas (Cubrinovski and McCahon, 2011). Comparison of the extent of liquefaction with the geological map of Forsyth et al. (2008) shows that most damage was caused on flood plain alluvium, 1st terrace gravels and young estuarine sand deposits, and less on young dune sand (beach) deposits.

The widespread occurrence of liquefaction could be observed from the large volumes of sand, silt, and water ejected from the ground surface immediately following the earthquake. 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 and a selection of these are presented in Figure 27. High groundwater was observed at one liquefied site (AMI Stadium) when investigated by Tonkin and Taylor in May 2011. In this area, the soil consists of alluvial silt of the Yaldhurst Member of the Springston Formation, with groundwater at 1.2 to >2.5m below surface. In Christchurch generally, the water table varies from about 5 m deep in the western suburbs to approximately 1.0 m to 1.5 m to the east of CBD Cubrinovsk and McCahon (2011). As the soils in this area are loose silty or sandy soils, this makes much of Christchurch highly susceptible to liquefaction during earthquakes.

Significant lateral spreading was evident in regions immediately adjacent to the Avon river, with large ground fissures observed at distances of up to 100m from the river bank in some instances. Volume loss due to liquefaction also resulted in permanent and often non-uniform ground settlement. This liquefaction-induced ground movement caused widespread damage to foundations in residential areas, as well as to a number of multi-storey buildings in the CBD. The primary modes of building response observed include tilting, settlement, and lateral movement – these often occurring in conjunction. In liquefied residential areas, much of the structural damage could be directly attributed to ground failure; thus, observations of this nature are reported in the building performance sections of this paper. Several case studies of multi-storey foundation performance in the CBD were also examined, and selected observations are presented below.

At the time of survey, only two of the buildings observed by the EEFIT team could be confirmed to have piled foundations. 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 28 shows the foundation beams of the building exposed by almost 300 mm, trapping the cars parked inside.

Another observed building (Figure 29) was confirmed to be on piles, though the depth of these piles was varying, with some quite shallow (Cubrinovski and McCahon, 2011). This building was one of two adjacent high-rise buildings titling significantly, shown in Figure 29a. 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 approximately the 3rd storey. Along the front of the piled building, a slight bulging of the pavement indicated that the foundation had experienced bearing failure on this side, which is consistent with the direction of tilt. The neighbouring building, with a basement foundation, was observed to have settled differentially due to settlement of the ground. The suspected relative vertical movement between the two buildings was confirmed by examining the skew of several wooden struts (originally horizontal), which connected the two buildings at the ground floor. 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 in this section, is being conducted by a researchers at the University of Canterbury (Giorgini et al., 2011).

Two examples were observed of high-rise buildings, reportedly on shallow foundations, that appeared to perform well. 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 30. While no information was available regarding the type of foundation of the building in Figure 30b, the 21-storey building in Figure 30a is reported to have a raft foundation on a non-uniform gravel layer (M. Cubrinovski, personal communication, 2011). Despite its apparent lack of settlement, at the time of survey it appeared that this building was tilting slightly. Interestingly, the apparent tilt was away from the river, in the direction of less observed liquefaction (hence our hesitation at categorically stating there was a tilt) and more sophisticated measurement is needed to confirm the magnitude of tilt, this example demonstrates how, in buildings of this size, the effects of permanent ground movement may be difficult to isolate from other factors when assessing the overall building performance.

Observations of Landslides

Landslides in the Port Hills area of Christchurch were triggered by the Darfield and Christchurch earthquakes and the July aftershock. On the 23 and 24th of February GNS Science flew reconnaissance flights over the Port Hills and Lyttleton and recorded 140 landslides that had affected at least 30km^2 (Hancox and Perrin, 2011). Rock falls caused five fatalities, affected roads and lifelines, and resulted in evacuation of approximately 100 homes. The EEFIT team visited several mass movement affected sites accompanied by GNS staff and observed failure of retaining walls, rockfalls, deep-seated landsliding in jointed volcanic rock, earth flows initiated by earth fall (in loess deposits) and cracks and rents. The team observed that ground within a distance of approximately 1.5 times the free standing cliff height was affected by rock fall hazard. The approximately 80m high coastal cliff at Sumner developed deep ground fissures after the February earthquake and subsequently failed during the magnitude M_w 6.3 aftershock in June 2011. Vertical and horizontal accelerations recoded in the port hills were very high. Rapid deployment of slope monitoring equipment by GeoNet (e.g. cGPS, ground pins and terrestrial laser scanning) enabled progressive failure to be monitored and the immediate risk to be managed. A typical example of a landslide is presented in Figure 31.

DISASTER MANAGEMENT IN RELATION TO BUILDING DAMAGE

On 23rd February, the Government declared a State of National Emergency applying specifically to the area under jurisdiction of Christchurch City Council. The State of National Emergency was extended on a weekly basis out to 30th April 2011 while vehicle recovery, structural assessment and demolition continue in some areas of the CBD. During this time the National Crisis Management Centre in Wellington was responsible for the overall response. Ministry for Civil Defence and Emergency Management (CDEM) coordinated logistical support and reports to Government.

In light of the concentration of damage within the CBD, the army and police established a cordon across the entire CBD (within 'the four avenues' – Bealey Avenue, Rolleston Avenue, Moorhouse Avenue and Fitzgerald Avenue) to prevent people from entering for their safety and to prevent looting. The CDEM established a temporary Emergency Operations Centre in the Christchurch Arts Centre to provide interagency coordination and emergency management in both response and recovery phases.

Once the Christchurch International Airport reopened for international flights on the 23rd February, Urban Search and Rescue (USAR) personnel arrived from overseas to assist local Civil Defence and USAR teams in rescue, victim recovery, debris clearance, and assessment, controlled demolition, shoring and stabilisation of damaged structures. Some 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).

After nearly two weeks this cordoned off area was divided into four green zones and a red zone. Residents and building owners were given controlled access to the green zones to collect belongings before they were reopened to the public. The red zone was continually reduced in size through 2011, but remains in place as at October 2011, with the area opened to the public for the first time for controlled bus tours (Bayer, 2011).

Within days of the earthquake New Zealand Aerial Mapping (NZAM) acquired 100mm aerial photography and LiDAR over the Christchurch City and Lyttelton areas to show building damage and geotechnical impacts. 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.

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 the fourth time the system has been used by New Zealand engineers following an earthquake (Gisborne 2007; Padang, Indonesia, 2009; and Darfield 2010) with refinements made to the system after each operation. The aggregated results are used to make decisions on controlling traffic, cordons, safe access corridors, and to indicate the economic impact of the earthquake (NZSEE, 2010).

Two levels of rapid evaluation were undertaken by teams of qualified engineers: Level 1 Rapid Assessment conducted by external inspection only; and Level 2 Rapid Assessment where a building is over two storeys and warrants further inspection (but is stable for entry) in the Level 1 assessment (NZSEE, 2010). The building inspection databases were maintained by the Christchurch City Council for reporting and analysis. Experiences from the 4th 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 allowing for detailed observation of earthquake damage to the interior of many different structures.

CONCLUSIONS

Although nearly all modern engineered structures survived the earthquake with relatively little structural damage, there are many lessons that can be learnt. The major ones 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. Observations collected on site raise the question of how retrofit programmes should be implemented and enforced, more so, considering that even the performance of many of the surveyed strengthened structures was not fully satisfactory. 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 instead a serious hazard to people in the proximity of buildings. Out-of-plane failures also mean that large portions of the original materials and finishes were lost, whereas the strengthening of historic buildings should pursue the preservation of a building as well as life safety Christchurch's community faces the difficult choice of whether to replace or save existing non-engineered masonry structures; this choice will depend on safety and financial factors, but also on the historical significance of each building. Whereas some structures are probably beyond repair, the ones deemed worthy of restoration could provide precious case studies to inform future retrofit policies.

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 was 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.

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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FIGURES

(a)

(b)

(c)

Figure 1– Damage to timber structures showing (a) a timber house that has "jumped" off its foundations; (b) a timber house with relatively little external structural damage to the timber, but collapse of external masonry chimney; (c) soft storey failure at the ground floor level of a two storey timber frame house.

(a) (b)

Figure 2– 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 3- 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).

a)

b)

Figure 4– Problems with structural irregularity. (a) and (b) TVNZ building with open street frontage and infilled frame at rear.

(a) (b)

Figure 5 - (a) Grand Chancellor Hotel building, showing overhang of driveway at ground floor level and (b) associated car park – seen on the right of the hotel.

(a) (b)

Figure 6– Moderate cracking in RC structural walls. (a) Cracks drawn on internal core wall in 7-storey building on Kilmore Street; (b) cracking at base of 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 7– Significant damage to 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.

Figure 8– (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 overall seating of 60 mm, and around 15 mm gap measured after earthquake. The original gap/seating allowance is not known.

(a) (b)

Figure 9– 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).

Figure 10– Eaton Place parking building. Punching shear failure in RC building.

Figure 11- 22-storey steel EBF/Moment Frame building, Pacific Tower, 2010.

(a) (b)

Figure 12 – (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

(a)

(b)

(c)

(d)

Figure 13 – Unreinforced masonry structures typologies: (a) two-storey 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.

(a) (b)

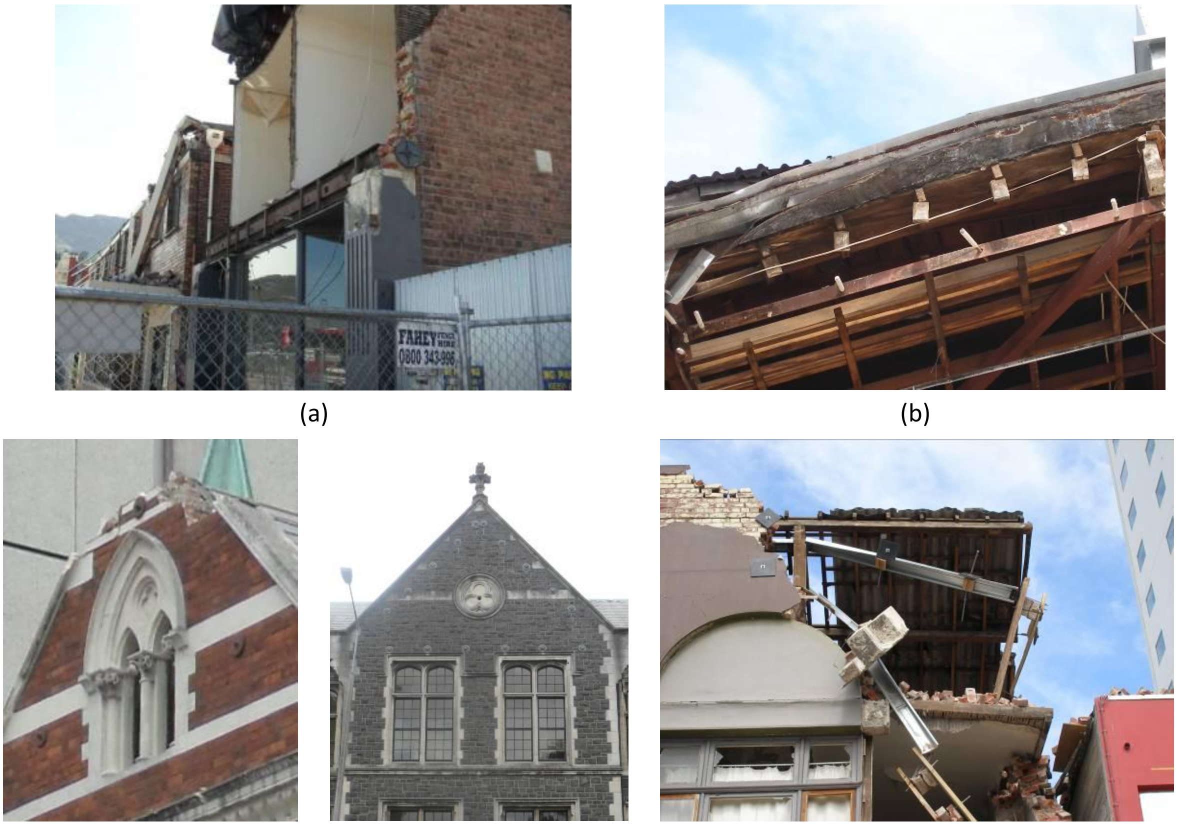
Figure 14 – Evidence of damage as consequence of the poor through-thickness connections of masonry walls: (a) complete lack of headers leading to separation of the two single-leaf panels of the wall; (b) small and far apart metallic profiles unable to ensure the unitary response of multi-leaf wall and prevent out-of-plane collapse of outer wythe.

(a) (b) (c)

(d) (e)

(f) (g) (h)

Figure 15 – Observed correlation between the quality of connections and damage mechanisms. Overturning of (a) whole or (b) portion of 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 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) (d) (e)

Figure 16 – 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.

(a) (b)

(c) (d)

Figure 17 – 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.

Figure 18 –Undamaged and lightly damaged RM buildings in the CBD.

(a) (b)

Figure 19 – 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) (b) (c)

Figure 20 – 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.

Figure 21 – 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.

(a) (b) (c)

Figure 22 – (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).

(a)

(b)

Figure 23 – (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 less than this. This level of deformation of the pavement 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 theme in the failure of roads and services due to lateral spreading.

Figure 24 – Stormwater outlet showing that large aggregate could be mobilised. The aggregate shown formed part of the sub-base of the road shown in the other figures.

(a) (b)

Figure 25 – (a) Abutment failure of the Anzac Drive bridge (built in 2000), caused by lateral spreading. The abutments of this bridge were 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)

Figure 26 – (a) Out of plumb telecommunication poles were common in areas of liquefaction. (b) Repair of buried services was commonly observed at the time of survey and this picture depicts a telecoms service being repaired.

a

b

Figure 27 - (a) Sand boils outside the AMI Stadium generated from liquefaction of the Yaldhurst Member of the Springston Formation during the February 2011 earthquake. (b) Typical sand boils observed in suburban areas of Christchurch

(a) (b)

Figure 28 – (a) and (b) Exposed foundation beams in a building with piled foundations.

(a) (b)

Figure 29 - (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.

(a) (b)

Figure 30 – (a) Localized settlement of the ground relative to a 20 storey high-rise building on a shallow raft foundation. This type of settlement was observed on all sides of the building. (b) Settlement of the ground relative to a large-plan multi-storey building with shallow foundations.

Figure 31 - Cargo containers have been 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.