PERFORMANCE OF MASONRY BUILDINGS DURING THE 2010 DARFIELD (NEW ZEALAND) EARTHQUAKE

Najif Ismail[1], Mike Griffith[2] and Jason M. Ingham[3]

Abstract

The M7.1 Darfield earthquake shook the town of Christchurch (New Zealand) in the early morning on Saturday 4th September 2010 and caused damage to a number of heritage unreinforced masonry buildings. No fatalities were reported directly linked to the earthquake, but the damage to important heritage buildings was the most extensive to have occurred since the 1931 Hawke's Bay earthquake. In general, the nature of damage was consistent with observations previously made on the seismic performance of unreinforced masonry buildings in large earthquakes, with aspects such as toppled chimneys and parapets, failure of gables and poorly secured face-loaded walls, and in-plane damage to masonry frames all being extensively documented. This report on the performance of the unreinforced masonry buildings in the 2010 Darfield earthquake provides details on typical building characteristics, a review of damage statistics obtained by interrogating the building assessment database that was compiled in association with post-earthquake building inspections, and a review of the characteristic failure modes that were observed.

*Keywords*: Earthquake, masonry, performance, damage, failure

[1] PhD Student, University of Auckland, Dept. of Civil & Environ. Eng., Private Bag 92019, Auckland 1142, New Zealand, nism009@aucklanduni.ac.nz
[2] Prof., University of Adelaide, School of Civil, Environ. & Min. Eng., North Terrace Campus, Adelaide, SA, 5005, Australia, mcgrif@civeng.adelaide.edu.au
[3] Associate Prof., University of Auckland, Dept. of Civil & Environ. Eng., Private Bag 92019, Auckland1142, New Zealand, j.ingham@auckland.ac.nz

Introduction

At 4.35 am on the morning of Saturday the 4th of September 2010 a magnitude 7.1 earthquake occurred approximately 40 km west of the city of Christchurch NZ at a depth of about 10 km (GNS 2010), having an epicentre located near the town of Darfield. The ground motion had a peak ground acceleration of about 0.25g and a spectral acceleration in the plateau region of about 0.75g that corresponds well with the design spectra for a site class D soil site in Christchurch for spectral periods greater than 0.2 seconds. In general, the earthquake represented 67-100% of the design level event, depending upon the spectral period being considered (see Fig 1), with most of Canterbury reporting damage consistent with MM8 on the Modified Mercalli intensity scale. The overall impression is that damage in the central business district (CBD) was reasonably contained, restricted primarily to unreinforced masonry (URM) buildings and damage to windows in taller steel and concrete structures. The absence of fatalities and more extensive damage is attributed to the comparatively high level of seismic design capability in New Zealand, and the fact that the CBD, which is the region containing the highest density of URM buildings, was almost completely unoccupied at 4.35 am. The soil conditions in Christchurch have three separate material types: river outwash gravels, sands, and marshy ground in former swamp areas, with the central city built mainly on gravels, although there are pockets of sand and soft soil in former marsh deposits. The earthquake ground motion characteristics shown in Figure 1 reflect the underlying soil condition, with the long period nature of the motion resulting from the soft soils upon which Christchurch is founded.
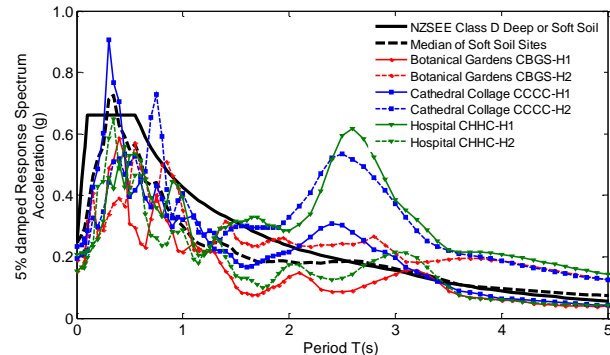
(a) Acceleration spectra
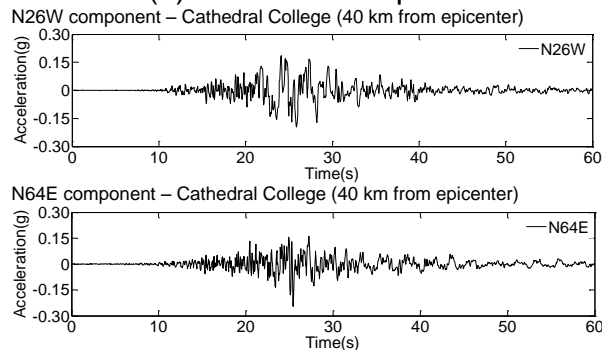
(b) Acceleration time-history records
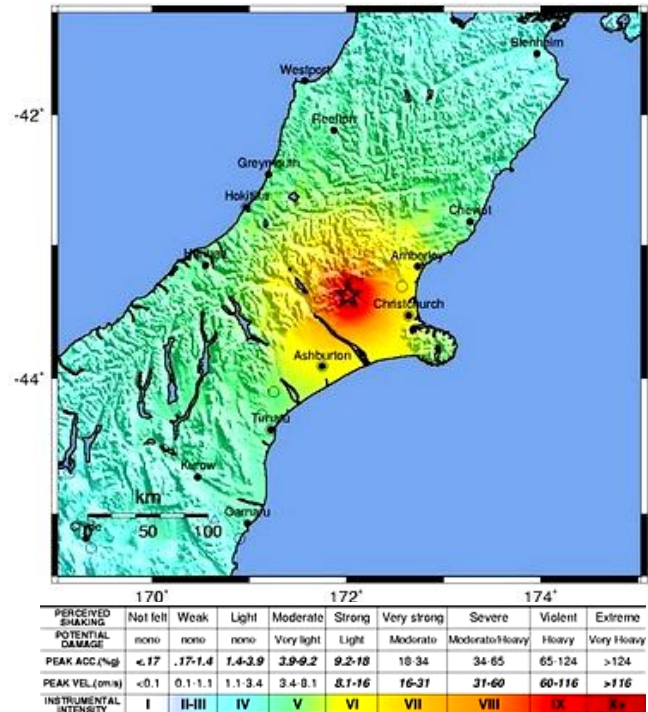
(c) US Geological Survey Shake Map

Figure 1. Details of earthquake ground motion [credit: GeoNet and USGS]

These soft soils effectively acted as a filter and removed high frequency ground motion (leading to smaller PGA values than on rock sites) but amplified long period motion, resulting in significantly larger longer period motion at about 2.5 second. As most URM buildings have a fundamental period of 0.2-0.3 seconds, the underlying ground conditions appear to have assisted in reducing the seismic demand in this period range to approximately 70% of the current NZS 1170.5 [SANZ 2004] design level loading. As can be seen in Fig 1b, strong ground shaking in the CBD had a duration of approximately 15 seconds with similar amplitudes in the two orthogonal recording directions. This lack of distinct directionality probably explains why parapet failures were observed in streets running in both the North-South and East-West directions.

Background

It is well established that URM buildings perform poorly in large magnitude earthquakes, with a brief selection of relevant prior earthquakes that have caused major damage to URM buildings being detailed below:

- The M7.8 1931 Hawke's Bay (New Zealand) earthquake caused widespread devastation to URM buildings in the city of Napier (see Figs 2a and 2b), with 256 fatalities. This earthquake remains the worst disaster of any type to occur on New Zealand soil [Dowrick 1998].
- The M5.6 1989 Newcastle (Australia) earthquake resulted in 13 fatalities and over 160 casualties. The earthquake damaged approximately 50,000 buildings (80% of these were homes) with unreinforced masonry buildings most widely affected [Page 1991] and to date is the only earthquake to have caused any fatalities in Australia (see Figs 2c and 2d).
- The M6.8 2007 Gisborne (New Zealand) earthquake caused damage to numerous unreinforced masonry buildings, including the collapse of 22 parapets [Davey and Blakie 2010]. Examples of damage to URM buildings are shown in Figs. 2e and 2f.
- The M8.8 2010 Maule (Chile) earthquake caused extensive damage to older houses, churches and other buildings constructed of unreinforced masonry or adobe, as shown in Figs 2g and 2h [EERI 2010].
- The M5.0 2010 Kalgoorlie-Boulder (Australia) earthquake occurred near the city of Kalgoorlie-Boulder, causing damage to many historic buildings (see Fig. 2i) and no fatalities [Edwards et al. 2010].

(a) The buildings destroyed by the 1931 earthquake and fires [Alexander Turnbull Library]

(b) Hastings Street, Napier after the 1931 earthquake [Alexander Turnbull Library]

(c) Out-of-plane wall failure in the 1989 Newcastle earthquake

(d) Parapet and cavity wall damage in the 1989 Newcastle earthquake

(e) Toppled parapet in the 2007 Gisborne earthquake

(f) Out-of-plane failure of a gable wall in the 2007 Gisborne earthquake

(g) 1940's URM house, Maipu Street, Concepcion, Chile

(h) Barros Arana Institute, Concepcion, Chile

(i) Damage to parapet and awning in Kalgoorie-boulder

Figure 2. Representative examples of damage to URM buildings in past earthquakes

Recent research has suggested that there are approximately 3750 URM buildings in New Zealand [Russell and Ingham 2010], with their distribution throughout New Zealand being aligned with the relative prosperity of communities during the period between approximately 1880 and 1930. Following the Darfield earthquake, it was reported that the Canterbury region had approximately 7600 earthquake prone buildings, with 958 of these buildings being constructed of unreinforced masonry [CCC 2010, Wells 2010].

Performance Observations

Post-earthquake inspection of building performance led to 595 URM buildings being assessed by Christchurch city council, out of the 958 URM buildings reported to exist in Christchurch. It is believed that the majority of non-assessed URM buildings were undamaged and outside the primary inspection zone associated with the CBD and arterial routes extending from the central city. General features of the 595 assessed URM buildings are reported in Figure 3. Figure 3c shows that the most common occupancy type was commercial or office buildings. The survey forms contained a field to record the estimated gross floor area of the building, and hence the estimated building footprint could be determined once accounting for the number of stories (see Fig 3b), but the data is unfortunately incomplete as only 301 entries were recorded for the 595 separate buildings assessed. It is not possible to establish from the database whether individual entries belonged to a stand-alone or a row building.
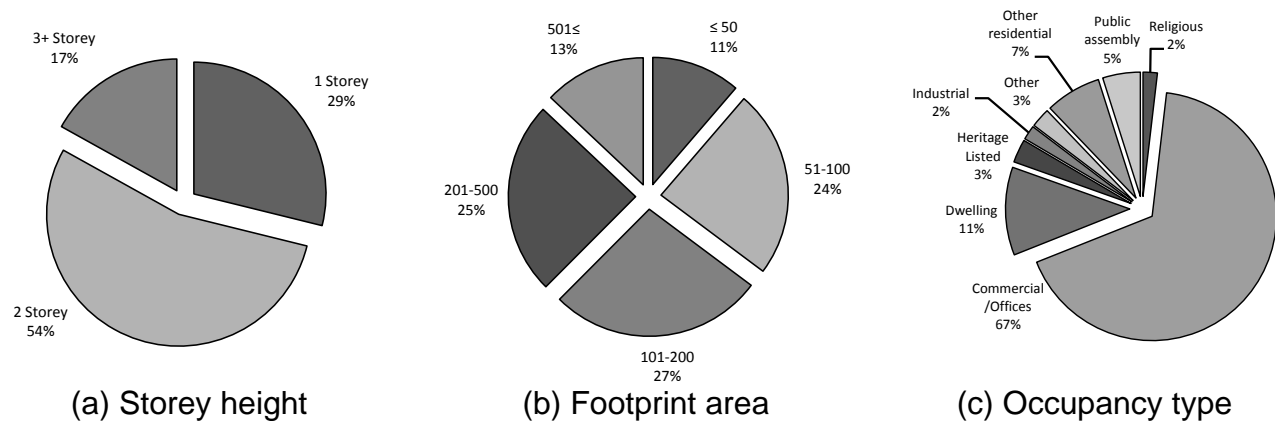
(a) Storey height   (b) Footprint area   (c) Occupancy type

Figure 3: Building characteristics derived from interrogation of the inspection database

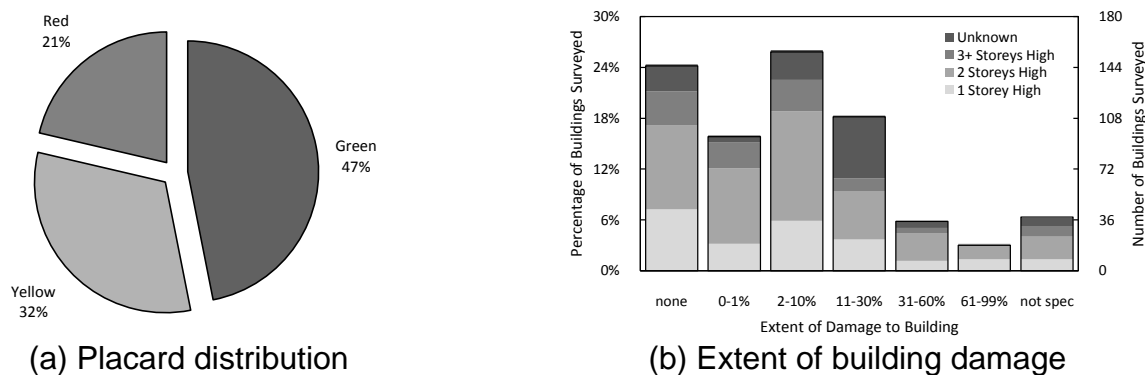(a) Placard distribution   (b) Extent of building damage

Figure 4: Building damage statistics derived from interrogation of the inspection database

In general, the observed damage to URM buildings in the 2010 Darfield earthquake was consistent with the expected seismic performance of this building form, and consistent with observed damage to URM buildings in past. Despite the known vulnerability of URM buildings to earthquake loading, 395 of the 595 buildings (66%) were rated as having 10% damage or less, with only 162 (34%) of the buildings assessed as having more than 10% damage (refer Figure 4). It was also possible to study the distribution of damage dependent on storey height, with the data indicating no definitive trend and a comparatively uniform level of damage assigned to buildings in each height category. The general observation from the debris of collapsed URM walls was that the kiln fired clay bricks were generally of sound condition, but that the mortar was in poor condition. In most cases the fallen debris had collapsed into individual bricks, rather than as larger chunks of masonry debris. However, it appears that superior mortar was often used in the ornate parapet above the centre of the wall facing the street, as this segment of the collapsed parapet often remained intact as it collapsed.

Chimney Collapse

Unsupported or unreinforced brick chimneys performed poorly in the earthquake (Figure 5(a)), with numerous chimney collapses occurring and many examples of badly damaged

chimneys. It was reported that one week after the earthquake, 14,000 insurance claims involving chimney damage had been received, from a total of 50,000 claims [NewstalkZB 2010]. The damaged chimneys pose a 'falling hazard' and were rapidly removed or restrained by emergency services personnel (see Figure 5(b)). In contrast, Figure 5(c) shows an example of a braced chimney that performed well.

| (a) Damaged chimneys | (b) Chimney removal | (c) Braced chimney |

Figure 5. Examples of chimney performance

Gable Wall Failure and Parapet Failure

Many gable end failures were observed, often collapsing onto or through the roof of an adjacent building (refer Figure 6). However, there were also many gable ends that survived, with the majority having some form of visible restraints that tied back to the roof structure.

| (a) 93 Manchester St | (b) 31 Kilmore street | (c) 40 Armagh street |

Figure 6. Examples of gable end wall failures

Numerous parapet failures were observed along both the building frontage and along their side walls, and for several URM buildings located on the corners of intersections, the parapets collapsed on both perpendicular walls (refer Figure 7). Awnings were typically supported using a tension rod tied back into the building front wall. Falling parapets typically landed on awnings, resulting in an overloading of the braces that supported these awnings and causing a punching shear failure in the masonry wall, identified by a crater (refer Figure 7(a)). Restraint of URM parapets against lateral loads has routinely been implemented since the 1940s, so whilst it is difficult to see these restraints unless roof access is available, it is believed that the majority of parapets that exhibited no damage in the earthquake were

provided with suitable lateral restraint. In several cases, it appears that parapets were braced back to the perpendicular parapet, which proved unsuccessful.

| (a) Awning failure | (b) Extensive parapet failure | (c) Collapse onto roof |

Figure 7. Examples of typical parapet failures

Out-of-plane Wall Failure

Out-of-plane failure was observed in numerous buildings and was attributed to poor or no anchorage of the wall to its supporting timber diaphragm. In the three instances shown in Figures 8(a), 8(b), and 8(c), it appears that the walls were not carrying significant vertical gravity loads, other than their self weight, due to the fact that the remaining roof structures appear to be primarily undamaged. In contrast, Figure 8(d) shows an out-of-plane failure of a side wall which was supporting the roof trusses prior to failure.
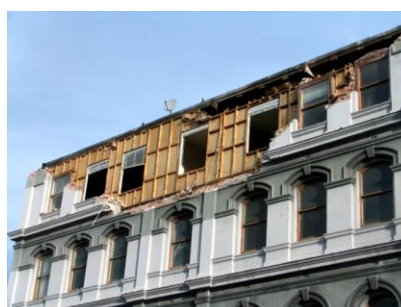

| (a)184 Worcester street | (b) 118 Manchester Street | (c) 179 Victoria Street |
| (d) Load bearing wall failure | (e) 140 Lichfield Street | (f) Dry-stacked masonry test |

Figure 8. Examples of typical out-of-plane wall failures

Several examples of face load wall failure (see Figure 8(e)) closely resembled observed damage (see Figure 8(f)) in dry stack masonry experiments [Restrepo-Velez and Magenes 2009], providing further support to the supposition that many of the wall failures were partly attributable to poor mortar strength. Cavity wall construction is generally believed to be much less common in New Zealand than is solid multi-leaf (or multi-wythe) construction. However, cavity wall construction can be extremely vulnerable to out-of-plane failure in earthquakes, particularly in situations where the cavity ties were poorly installed, or more commonly have corroded over time, as the wall is then comparatively slender and less stable than for solid construction. Figures 9(a) and (b) show examples of cavity wall buildings that suffered out-of-plane wall failures. Figure 9(c) shows that cavity ties were present but were insufficient to prevent the outer leaf from failing.

(a) Cavity wall failure   (b) Cavity wall failure in CBD   (c) Insufficient wall ties

Figure 9. Examples of typical out-of-plane cavity wall failures

A typical wall-to-diaphragm (roof or floor) anchor typically consists of a long 20 mm bolt with a large circular disk of about 150-200 mm diameter between the wall exterior and nut that clamped the disk to the wall. In some cases wall-diaphragm anchors remained visible in the diaphragm after the wall had failed (refer Figure 10(a)), again indicating that failure had occurred due to bed joint shear in the masonry. It was observed in some buildings that the anchor successfully prevented the restrained wall from failing, but was not able to prevent toppling of the parapet that was located above the anchor. A significant feature of the earthquake was the number of occasions where anchored walls performed well during the earthquake (see Figure 10 (b) and (c)).
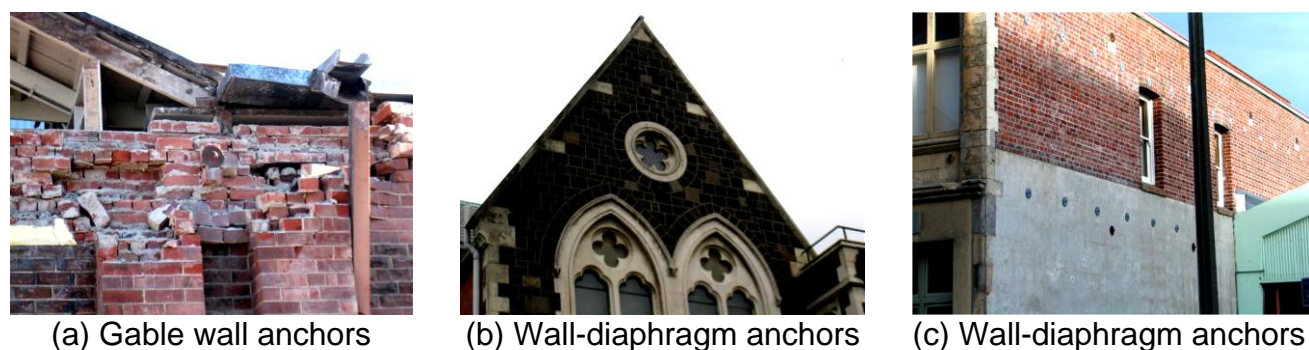

(a) Gable wall anchors   (b) Wall-diaphragm anchors   (c) Wall-diaphragm anchors

Figure 10. Examples of insufficient and successful anchors

In-plane Wall Failure

Where walls exhibited some damage to in-plane deformation the cracks were mostly seen to pass vertically through the lintels over door or window openings (refer Figure 11). In some cases, diagonal shear cracks in piers (refer Figure 12(a)) were also observed and were attributed to the relative geometric characteristics of the pier to the spandrels of the wall. Figures 12(b) shows a 7-storey office building that is reported to consist of load bearing masonry and is the most significant masonry building, at least in terms of height, in Christchurch. The bottom two stories are reported to be reinforced concrete while the top five stories are reported to have load bearing unreinforced masonry piers around the exterior of the building and a steel frame internally (columns spaced roughly at 5 m) with timber floors throughout [NZHPT 2010]. The masonry piers, having dimensions of approximately 1200 x 900 mm, were badly cracked at levels 3 and 4. Further inspection by the assessment team exposed the internal face of one pier on the western face of the building to reveal that the external cracking continued through the entire pier thickness. After considerable community discussion, including recognition that surrounding buildings were required to remain vacated due to the potential for building collapse, it was decided that the building would be demolished. St Elmo Chambers is also a 7-storey building, reported to be a reinforced concrete frame building with external clay brick masonry piers. Owing to the absence of control joints between the masonry and concrete frame, X shape cracks suggest that the piers failed in diagonal shear (refer Figure 12(c)).

| (a) vertical spandrel crack | (b) Vertical spandrel crack | (c) Diagonal spandrel crack |

Figure 11. Examples of in-plane wall damage above window openings

| (a) 121 Worcester Street | (b) 160 Manchester Street | (c) St Elmo Chambers |

Figure 12. Examples of piers damaged with in-plane shear cracking

Partial Wall Failure

Another interesting feature of this earthquake is the observation of walls that only partly failed, allowing for identification of the specific failure mode at its onset. The first example is a 2-storey URM building on Ferry Road (see Figure 13) where the front facade of the building had started to fail out-of-plane despite the presence of wall-roof diaphragm anchors. As is shown, the anchors were on the verge of pulling through the masonry wall. Internal inspection of the building revealed that the front wall had separated from the long side walls of the building and moved approximately 50 mm towards the road with respect to the ceiling/roof diaphragm (Figure 13(c)).

| (a) Building overview | (b) Partial anchorage failure | (c) Separation of wall |

Figure 13. Example of wall-roof anchor failure leading to partial collapse of the wall

It is believed that due to the nature of strength degradation of the brickwork at the onset of a punching shear failure, the anchorage had effectively failed and offered little residual resistance against further shaking. The only reason for why the wall did not completely collapse is probably due to the earthquake not imposing sufficient displacement on the wall after the anchorage failure. A similar style of partial failure was observed in another building (refer Figure 14(a)) but the building was observed externally only. An example of a gable end partial failure is shown in Figure 14(b). There were frequent examples of wall-diaphragm anchors that had deformed plastically or alternatively were inadequately fixed (refer Figure 14(c) where a circular plate can be seen to be slack due to plastic stretching of the anchor rod).

| (a) Anchor failure | (b) Gable anchor failed | (c) Yielded anchor |

Figure 14. Examples of wall-roof anchor failure leading to partial collapse of the wall

Return Wall Separation and Pounding Damage

Many buildings exhibited substantial cracking between their front wall and side (return) walls (refer Figure 15(a) and (b)). This damage is not necessarily a catastrophic problem if stiff horizontal diaphragms are well connected to the walls in both directions, but when diaphragm connectivity is not sufficient, complete out-of-plane collapse of one or both walls may occur. Several instances of damage due to buildings pounding against each other during the earthquake were observed. Figure 15(c) shows how the shorter building in the centre, which has different floor heights than the building to the left, damaged the column of the taller building at its top storey.

(a) Wall separation   (b) Wall separation   (c) Overview and close up of pounding

Figure 15. Examples of roof wall separation and pounding damage

Building Damage due to Ground Deformation

As shown in Figure 16, several cases of extreme ground deformation were observed, and there were numerous cases where large crack widths formed in residential timber framed structures having a masonry veneer (Figure 16(b)). There were also cases where ground liquefaction had resulted in masonry structures having sunk into the ground (Figure 16(c)).

(a) Cracking   (b) Veneer damage   (c) Liquefaction

Figure 16. Examples of building damage due to ground deformation

Closing Remarks

On the few occasions that building owners or occupants were in attendance it was possible to gain access to the interior of URM buildings and often observe that some separation had occurred between the floor and/or roof diaphragms and the masonry walls (in the out-of-plane direction). There were many instances of buildings that were structurally sound themselves but had suffered damage or were yellow or red-tagged owing to 'falling hazards' from neighbouring buildings. In some instances it was clear that a parapet or chimney from a neighbouring building had fallen onto or through the roof, being the only damage to the structure. These examples of 'collateral damage and risk', and the associated business interruption costs, will surely make the financial impact of this earthquake much greater than just the cost of rebuilding.

Acknowledgements

References

GNS 2010: GNS "Darfield earthquake damages Canterbury", http://www.geonet.org.nz, 16 Sept 2010; retrieved 16 Sept 2010.

SANZ 2004: Standards New Zealand. "NZS 1170.5.04: Structural Design Actions- Part 5: Earthquake Actions". Wellington, New Zealand, 2004.

Dowrick 1998: Dowrick, D. "Damage and Intensities in the Magnitude 7.8 1931 Hawke's Bay, New Zealand, Earthquake". Bulletin of the New Zealand Society for Earthquake Engineering, 30(2), 1998, pp. 133-158.

Page 1991: Page, A. "The Newcastle Earthquake – Behaviour of Masonry Structures". Masonry International (British Masonry Society), 5(1), 1991, pp. 11-18.

Davey and Blaikie 2010: Davey, R. A., and E. L. Blaikie "Predicted and Observed Performance of Masonry Parapets in the 2007 Gisborne Earthquake". Proceedings of the New Zealand Society for Earthquake Engineering Conference, Wellington, New Zealand, 26-28 March 2010.

EERI 2010: EERI. "The Mw 8.8 Chile Earthquake of February 27, 2010". Earthquake Engineering Research Institute Newsletter, http://www.eeri.org, June 2010, retrieved 16 Sept 2010.

Edwards et al. 2010: Edwards, M., M. Griffith, M. Wehner, N. Lam, N. Corby, M. Jakab and N. Habili "The Kalgoorlie Earthquake of the 20th April 2010: Preliminary Damage Survey Outcomes". Proceedings of the Australian Earthquake Engineering Society Conference, Perth. Australia, 26-28 November 2010.

Russell and Ingham 2010: Russell, A.P. and J. M. Ingham "Prevalence of New Zealand's Unreinforced Masonry Buildings". Bulletin of the New Zealand Society for Earthquake Engineering, 43(3), 2010, pp. 182-201.

CCC 2010: Christchurch City Council "Review of Earthquake-prone, Dangerous and Insanitary Buildings Policy". http://www1.ccc.govt.nz , 4 March 2010, retrieved 20 Sept 2010.

Wells 2010: Wells, S. "Council Fast Tracks Earthquake Policy". http://www.stuff.co.nz, 15 Sept 2010, retrieved 15 Sept 2010.

NewstalkZB 2010: NewstalkZB "Most Damage Claims over $10k", http://www.newstalkzb.co.nz, 13 Sept 2010, retrieved 13 Sept 2010.

Restrepo-Velez and Maganese 2009: Restrepo-Velez, L.F. and G. Magenes "Static Tests on Dry Stone Masonry and Evaluation of Static Collapse Multipliers". Research Report No. ROSE-2009/02, IUSS Press, Fondazione EUCENTRE, Pavia, Italy, 2009, pp. 72.

NZHPT 2010: New Zealand Historic Places Trust "The Manchester Courts". http://www.historic.org.nz, Retrieved 9 Sept 2010.