



Seismic performance of Reinforced Concrete Frame with Masonry Infill buildings in the 2010/2011 Canterbury, New Zealand earthquakes

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Abstract

As a result of the 2010/2011 Canterbury, New Zealand earthquakes, Reinforced Concrete Frame with Masonry Infill (RCFMI) buildings experienced a level of damage that was different to that observed for other construction systems. An extensive survey was conducted by the Christchurch City Council and the Canterbury Earthquake Recovery Authority following the earthquakes to document damaged buildings in the affected area. The collected data were then merged into the Canterbury Earthquake Building Assessment (CEBA) database, and the database was utilised to assess the damage sustained by RCFMI buildings. In order to provide a reliable estimation of the seismic vulnerability for RCFMI buildings in the region, empirical fragility curves were generated using the Lognormal Cumulative Distribution method by utilising the post-earthquake dataset provided in the CEBA database, with the expected median and standard deviation values derived using the Maximum Likelihood Estimation method. Results showed that the majority of low-rise to mid-rise RCFMI buildings performed satisfactorily during the Canterbury earthquakes, with several high-rise RCFMI buildings sustaining moderate to heavy damage.

Keywords Empirical fragility curves · Masonry infill · Post-earthquake assessment

1 Introduction

In 2010 and 2011 the Canterbury region of New Zealand experienced a series of earthquakes, with two major earthquakes having a significant impact to buildings in the earthquake affected region. On 4 September 2010 the M_w 7.1 Darfield earthquake shook the region with an epicentre located at a depth of 11 km and centred approximately

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40 km west of central Christchurch (GeoNet 2014b). Subsequently, the M_w 6.3 Lyttelton aftershock earthquake (also commonly referred to as the 2011 Christchurch earthquake) struck on 22 February 2011 with an epicentre having a shallow depth of 5 km and located 10 km south-east of central Christchurch (GeoNet 2014a). The aftershock earthquake generated significant ground motion intensities that resulted in moderate to severe damage to many buildings in the densely-populated Christchurch area, with some of the buildings completely collapsing (Bradley and Cubrinovski 2011). The 2010/2011 earthquake sequence in the region is hereafter referred to as the Canterbury earthquakes.

Following the Canterbury earthquakes it was observed that unreinforced masonry (URM) buildings sustained an extensive level of damage and generally performed poorly, with the inadequate seismic performance of URM buildings in Christchurch having been thoroughly reported elsewhere (Dizhur et al. 2010, 2011; Giarretton et al. 2016; Ingham and Griffith 2010, 2011; Moon et al. 2014; Senaldi et al. 2012, 2015). The seismic performance of reinforced concrete (RC) buildings in Christchurch was also well documented, with reports by Kam et al. (2010) showing evidence that RC buildings, in most cases, performed satisfactorily in the 2010 Darfield earthquake. However, the 2011 Lyttelton earthquake resulted in extensive damage to several RC buildings, with some existing non-ductile and modern seismically-designed RC buildings experienced major damage (Kam et al. 2011).

In contrast to URM and RC buildings, the seismic performance of Reinforced Concrete Frame with Masonry Infill (RCFMI) buildings in the Canterbury earthquakes has not previously been specifically reported, despite the fact that these buildings constituted a large proportion of existing buildings in the Christchurch region. Prior to the Canterbury earthquakes, it was thought that RCFMI buildings might be vulnerable to earthquake induced shaking because some RCFMI buildings were constructed prior to the introduction of New Zealand's first seismic code in 1935 (NZS 95 1935). Therefore, the study reported herein addresses the seismic performance assessment of commercial RCFMI buildings following the Canterbury earthquakes and estimation of the seismic vulnerability of these buildings in the region.

In order to assess the seismic performance of RCFMI buildings following the Canterbury earthquakes, the Canterbury Earthquake Building Assessment (CEBA) database was acquired from Geological and Nuclear Science (GNS). The CEBA database was developed based on post-earthquake data collected by Christchurch City Council (CCC), the Canterbury Earthquake Recovery Authority (CERA), and Tonkin and Taylor Limited that exclusively provided land damage information (Lin et al. 2016). The CEBA database provides information on damaged buildings, such as building addresses, the number of storeys, the year of construction, the lateral-load resisting system, infill types, presence of cavity construction, category of building damage placards, the current state of buildings, peak ground acceleration (PGA) values, and other related information.

10,777 damaged buildings were documented in the CEBA database following the Canterbury earthquakes, with many of these buildings being residential buildings (6062 out of 10,777; 56%). 3528 buildings (33%) in the CEBA database were categorised as commercial buildings, and 191 buildings of the 3528 (5%) commercial buildings were classified in the database as RCFMI buildings (Lin et al. 2016). The term 'commercial buildings' as used herein is defined as buildings considered as places of business, such as retail and office buildings. It is suspected that this estimated number of RCFMI buildings in the Christchurch region is low, with an earlier report from Kam et al. (2011) describing 209 RCFMI buildings in the Christchurch Central Business District (CBD) alone. It is also assumed that some RCFMI buildings were assigned an incorrect building type or were not

surveyed because they exhibited no damage. However, no effort was made in this study to revise the building type entries in the CEBA database.

It was also recognised that in some cases the information in the CEBA database that related to building characteristics and to building damage assessments was incomplete. Hence, a sidewalk survey was conducted in the region in order to compile key missing details for the 191 identified RCFMI buildings. In addition to the sidewalk survey, efforts were made to acquire post-earthquake documentation from CCC and Land Information New Zealand (LINZ).

The collected data from the sidewalk survey was cross-referenced with the CEBA database, and the damaged buildings were assigned to damage states. Building damage was classified into the categories of structural damage, possibility of collapse or partial collapse, building leaning, overhead falling hazard from components of damaged buildings, geotechnical damage, potential hazard from neighbouring buildings, and non-structural damage. Interpretation of structural damage accounted for structural components such as columns, beams, beam-column joints, and masonry infill walls. The extent of damage for each component was categorised as None/Minor, Moderate, or Severe. For example, hair-line cracks on structural columns or beams were categorised as Minor damage, while longitudinal reinforcement buckling was classified as Severe damage. In order to determine the overall state of damage for each building, the mean damage ratio of structural components was calculated based on the percentage of structural components with different damage extents (see Lin et al. 2014). The definitions adopted for these damage states was influenced by the usability category that had been assigned in the Level 2 detailed seismic assessments conducted by CCC, where usability was classified as: occupiable with no investigation required (G1); occupiable with repairs required (G2); short term entry permitted (Y1); no entry to parts of building until repaired or demolished (Y2); significant damage (R1); severe damage (R2); and risk coming from adjacent buildings or ground failure (R3) (Lin et al. 2016). The damaged RCFMI buildings were assigned to damage states of Slight (DS1); Light (DS2); Moderate (DS3); and Heavy (DS4), which are consistent with ATC-20 and EMS-98 (Grünthal 1998) damage states. It is noted that the Major (DS5) damage state was not considered in this study as no RCFMI buildings collapsed during the Canterbury earthquake. Despite ATC-20 and EMS 90 (Grünthal 1998) also delineating the damage state into five categories, the different methodologies adopted by CCC, ATC-20 and EMS-20 potentially results in significant variation when deriving fragility curves.

Based on the compiled post-earthquake dataset, a Damage Probability Matrix (DPM) was derived to describe the effect of ground motion intensities on RCFMI building damage. Finally, empirical fragility curves for the defined damage states were developed using the Lognormal Cumulative Distribution method to estimate the earthquake vulnerability of RCFMI buildings in the region. The proposed fragility curves in this study were also compared with previous empirical fragility curves generated by utilising post-earthquake datasets from different countries, such as following the 2009 L'Aquila, Italy earthquake, to compare the observed vulnerability of RCFMI buildings during past earthquakes.

2 Survey of damaged RCFMI buildings

Figure 1 shows the location of damaged RCFMI buildings in the Christchurch region in conjunction with the building placard status assigned from Level 1 assessments conducted by CCC following the 2011 Lyttelton earthquake (NZSEE Study Group 2002). Green

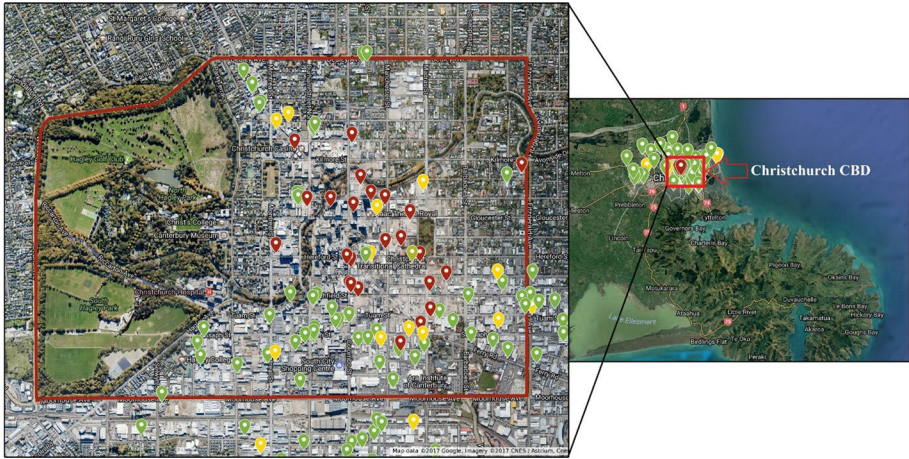


Fig. 1 Location of 191 damaged RCFMI buildings in Christchurch documented in the CEBA database. Green, occupiable; yellow, restricted entry; red, no entry

placards indicate that the buildings were inspected and determined to be safe for reoccupation based on brief exterior inspection, yellow placards represent buildings that were inspected and assigned restricted entry, and red placards indicate buildings that were considered unsafe and likely to have moderate to major damage.

In general, RCFMI buildings exhibited good seismic performance during the Canterbury earthquakes, as the majority (129 out of 191 buildings; 68%) of the observed buildings were assigned green placards. It is noted that RCFMI buildings that were assigned green placards in the CEBA database exhibited insignificant to minor damage, with little attention given to undamaged buildings (Lin et al. 2016). It is also recognised that a large number of additional RCFMI buildings were observed in Christchurch during the sidewalk survey that were not included in the CEBA database. These non-surveyed buildings potentially result in significant coverage error in the building database, and this error can lead to a bias in the building damage statistics. According to Rossetto et al. (2013), the sampling bias can be treated by assuming that non-surveyed buildings were undamaged, and that the building stock can be completed using census data available in the region. However, the census building data in the Canterbury region were again not available, and therefore the RCFMI buildings reported herein were identified exclusively based on the damaged RCFMI buildings available in the CEBA database. Because the non-surveyed buildings presumably exhibited no damage, the proportion of buildings meriting green placards may well be significantly higher than 68%. Moreover, a comparatively small number of RCFMI buildings were assigned yellow placards (45 out of 191 buildings; 24%) and red placards (17 out of 191 buildings; 8%), and these proportions would be further reduced when accounting for the potential sampling bias.

From an analysis of the current state of the 191 RCFMI buildings as documented on 31 October 2016 it was identified that 49 out of 191 RCFMI buildings (26%) were demolished, while the majority (74%) of these buildings remained standing (see Fig. 2a). A high proportion of the RCFMI buildings that were demolished following the 2011 Lyttelton earthquake were initially assigned a red placard (14 demolished buildings out of 17 with red placard; 82%) or a yellow placard, (27 demolished buildings out of 45 with yellow

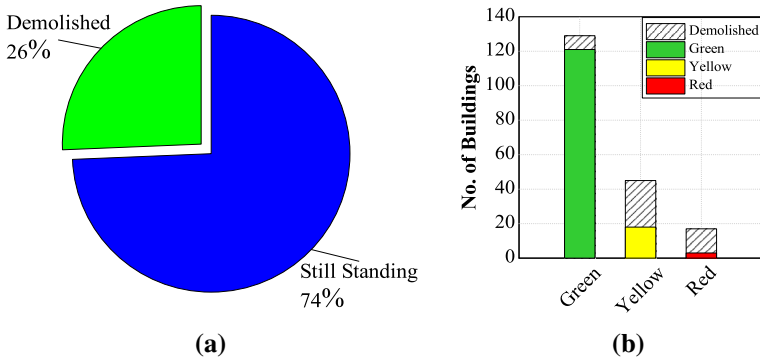


Fig. 2 Demolition statistics for RCFMI buildings as documented on 31 October 2016. **a** Current state of buildings, **b** proportion of demolished building for each building placard type

placard; 60%), as presented in Fig. 2b, whereas only a small number of demolished RCFMI buildings were initially assigned to green placards, (8 demolished out of 129 buildings with green placard; 6%), despite the fact that these buildings exhibited insignificant to minor damage. The cost of repairing and strengthening these buildings to the minimum national standard of required earthquake capacity is likely to have been the dominant factor dictating building demolition, with stakeholders perhaps deeming that replacement with a new building was more cost effective (King et al. 2014).

3 Building characteristics

The masonry infill wall materials of the surveyed RCFMI buildings were documented in the CEBA database. It was shown that the use of concrete block as infill was more prominent (68%) compared to clay brick as infill (26%), as shown in Fig. 3a. It is remarked that 12 (6%) damaged RCFMI buildings have no infill materials documented due to the presence of plaster covering the masonry wall.

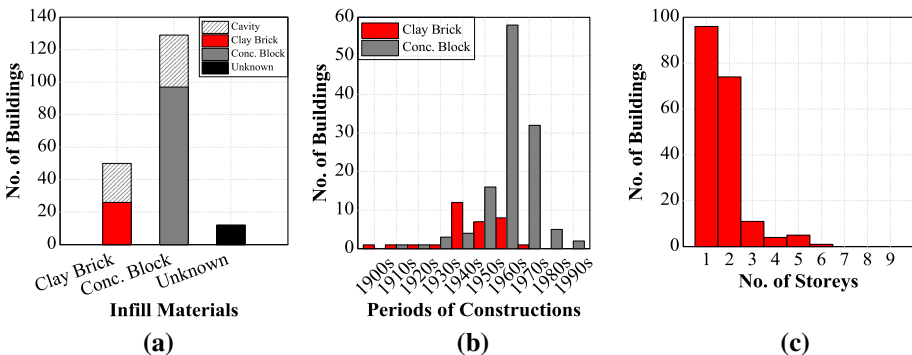


Fig. 3 RCFMI building attributes. **a** Type of infill material, **b** year of construction, **c** number of stories above grade

Fig. 4 An example of an RCFMI building having masonry cavity infill wall construction

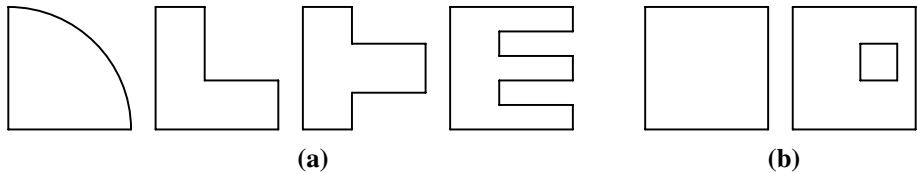
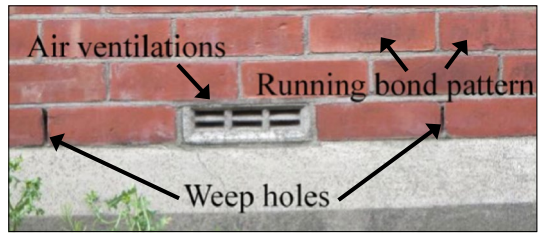


Fig. 5 Building footprint irregularity. **a** Irregular footprint, **b** regular footprint

The morphology of masonry infill walls as being constructed of either solid or cavity-wall was considered due to the potentially significant effect of this parameter on building performance during the earthquakes. Cavity-wall construction can be defined as a form of masonry infill wall consisting of two leaves of masonry walls that are separated by an air cavity and in some cases, are interconnected by a tie system. The RCFMI buildings having cavity-wall construction were identified based upon the visible presence of air vents, weep holes and/or a running bond pattern of the masonry infill wall as presented in Fig. 4. In total, it was observed that 56 buildings (29%) of the 191 damaged RCFMI building had cavity-wall construction, with 24 buildings having clay brick cavity-wall construction and 32 buildings having concrete block cavity-wall construction.

The type of infill material was found to be strongly associated with the period of construction of the RCFMI buildings. As can be seen in Fig. 3b, clay brick infill buildings were initially constructed in the early 1900s, and there was an elevated level of construction of this building type from the 1940s to the early 1960s. However, the use of concrete block as infill substantially increased in the 1950s, and commencing in the late 1950s, concrete block fully replaced the use of clay brick as an infill material in the region.

Figure 3c shows the number of stories for the surveyed RCFMI buildings in the Christchurch region. The majority (95%) of surveyed buildings were low-rise (1–3 storeys), where 1-storey buildings constituted the greatest proportion (50%) of the total 191 observed RCFMI buildings. 2-storey buildings had a proportion of 39% and 3-storey buildings accounted for 6% of the total surveyed RCFMI buildings. It should be considered that the RCFMI construction type was generally not used in New Zealand for mid to high-rise buildings, explaining the small number of buildings (5%) having 4+ storeys.

RCFMI buildings with footprint irregularity were considered herein because it was suspected that buildings with irregular footprint might be more susceptible to torsional effect when subjected to earthquake induced shaking. RCFMI buildings with various footprint were assigned to two general classes as having either irregular or regular footprints (see Fig. 5). RCFMI buildings with regular footprints were defined based on the distribution of the lateral load resisting elements such as columns, beams and walls, and having a relatively rectangular and regular shape. As identified during the sidewalk survey, RCFMI

buildings having a regular footprint were commonly observed (143 out of 191 buildings; 75%) compared to RCFMI buildings with an irregular footprint (48 out of 191 buildings; 25%).

4 Observed failure modes

The level of damage and failure modes of damaged RCFMI buildings were recorded following the Canterbury earthquakes, and were well-presented in the CEBA database. In this study, failure modes of the reinforced concrete frame and/or masonry infill walls were classified into different categories, with descriptions of the commonly observed failure modes summarised in Table 1.

The most common failure modes observed for the masonry infill walls were in-plane failures (35 out of 191 buildings; 18%), followed by out-of-plane failures (12 out of 191 buildings; 6%). Meanwhile, the general failure modes observed for RC frames were joint failure between RC beams and columns (22 out of 191 buildings; 12%), with some RCFMI buildings sustaining flexural yielding (19 out of 191 buildings; 10%) and shear failures (17 out of 191 buildings; 9%) in RC columns.

5 Determination of damage states

Damage states are a distinct level or scale that are used in seismic vulnerability assessment to assign the level of building damage observed following an earthquake. In this study, damage states were defined based on the usability category determined from Level 2 detailed seismic assessments documented by CCC after the 2011 Lyttelton earthquake. It is noted that there was no damage documentation of RCFMI buildings following the 2010 Darfield earthquake and the information presented hereafter mainly focuses on the building damage following the 2011 Lyttelton earthquake. Initially, CCC conducted Level 1 rapid assessments that resulted in the level of damage being assigned to three building placard categories of green, yellow, and red on the basis of estimated overall building damage observed from the building exterior.

Table 2 shows the classification of damage states considered herein, with the number of observed damaged buildings associated with each damage state being presented in Fig. 7. It is also noted that there were no reports of RCFMI buildings having collapsed during the Canterbury earthquakes, such that the collapse damage state (DS5) was not considered herein.

Figure 8 illustrates the damage sustained by infill material and the correlation between wall morphology and building damage following the 2011 Lyttelton earthquake. In general, it was observed that clay brick was the more vulnerable infill material in comparison with concrete block. However, there was no clear indication that wall morphology specifically affected the performance of buildings in the 2011 Lyttelton earthquakes.

Clay brick infill exhibited a reasonably similar performance in the 2011 Lyttelton earthquake for both solid and cavity-wall construction, with 50% of the total surveyed clay brick RCFMI buildings sustaining moderate to heavy damage (DS3–DS4). Despite the fact that 40% of the concrete block RCFMI buildings having cavity-wall construction experienced moderate to heavy damage (DS3–DS4), the majority of RCFMI buildings (82%)

Table 1 Observed failure modes in the Canterbury earthquakes

Failure modes	RCFMI building component	Descriptions
Diagonal tension (in-plane)	Masonry infill walls	Presence of diagonally oriented cracks typically in an x-shape (Fig. 6a, c)
Out-of-plane		Failure of masonry wall due to out-of-plane deformations (Fig. 6b)
Shear failure	Reinforced concrete (RC) frame	Diagonal cracking of RC columns and beams (Fig. 6a)
Flexural yielding		Horizontal cracking of RC columns and beams due to yielding of flexural reinforcement (Fig. 6d)
Joint failure		Significant cracking in beam-column joints (Fig. 6e)

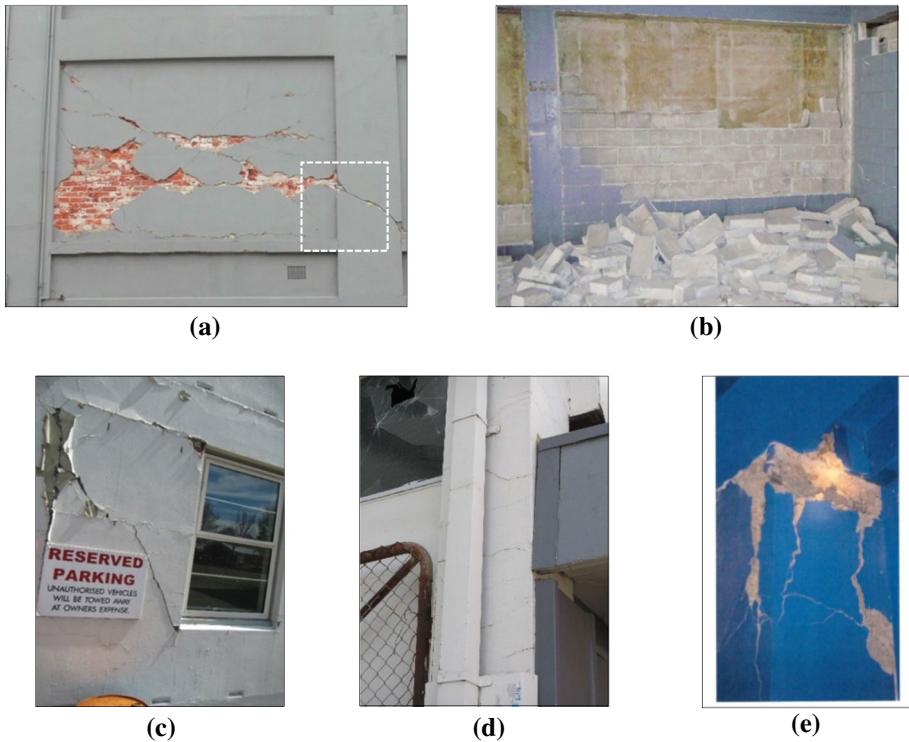


Fig. 6 Observed failures modes for RCFMI building components in the Canterbury earthquakes. **a** In-plane failure of clay brick infill and shear failure of RC column (highlighted in box), **b** out-of-plane failure of concrete block infill, **c** shear cracking of infill due to presence of opening, **d** flexural yielding of RC column, **e** RC joint failure

constructed using solid concrete block as infill performed satisfactorily during the 2011 Lyttelton earthquake.

RCFMI buildings with a regular footprint exhibited more severe damage in the 2011 Lyttelton earthquake compared to RCFMI buildings having an irregular footprint (see Fig. 9). It was observed that 39% of RCFMI buildings with a regular footprint sustained moderate to heavy damage (D3 to D4), whereas only 10% of RCFMI buildings with an irregular footprint (10%) sustained moderate to heavy damage. This finding is contrary to the widely understood effect where building geometric irregularity negatively influences building earthquake performance, and suggests that building footprint geometry did not significantly contribute to the observed damage characteristics of RCFMI in the 2011 Lyttelton earthquake.

Ground instability observed in the 2011 Lyttelton earthquake included liquefaction (silt ejection), lateral spreading, and/or differential settlement that potentially affected foundation subsidence (Cubrinovski et al. 2011). In Fig. 10 the different levels of damage to RCFMI buildings following the 2011 Lyttelton earthquake due to the influence of ground instability are shown. It was observed that 95 of the total surveyed 191 RCFMI buildings exhibited various level of damage generated by the combined effects of ground instability and earthquake shaking, with 20 out of 95 (21%) buildings sustaining moderate to major

Table 2 Definition of damage states

Damage States	Level 2 assessment	Estimated building damage (%)	Description
DS1—Slight	G1	0–1	Insignificant damage, and the buildings can be occupied immediately
DS2—Light	G2	2–10	Minor damage, with repair or strengthening required, but the buildings can be occupied immediately
DS3—Moderate	Y1	11–30	Moderate damage, with repair or strengthening required, and only short-term entry allowed to some parts of buildings
DS4—Heavy	Y2; R1; R2; R3	31–100	Heavy damage, with repair or strengthening required, and some parts of buildings having restricted entry until secured or demolished

Fig. 7 Damage state assessment for RCFMI buildings

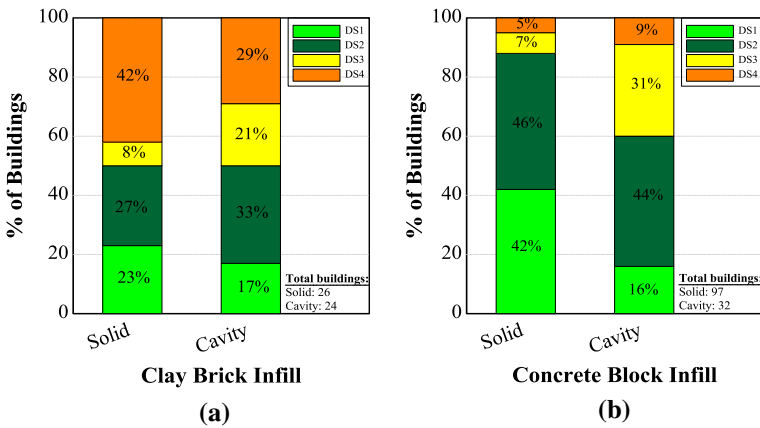
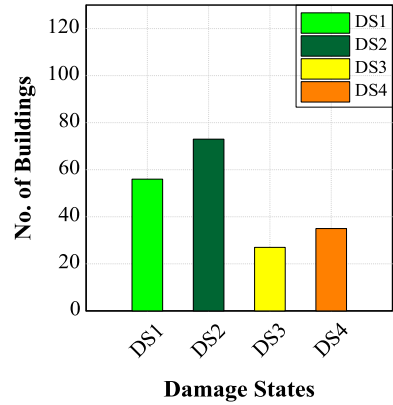


Fig. 8 Extent of damage to infill material. **a** Clay brick, **b** concrete block

Fig. 9 The influence of building footprint geometry on damage to RCFMI buildings

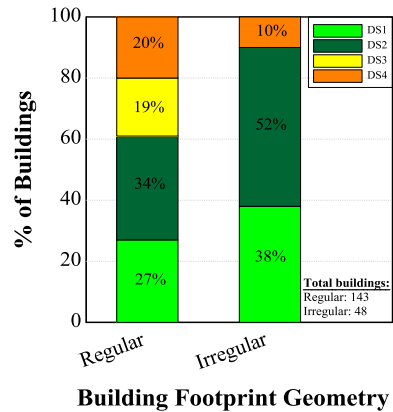
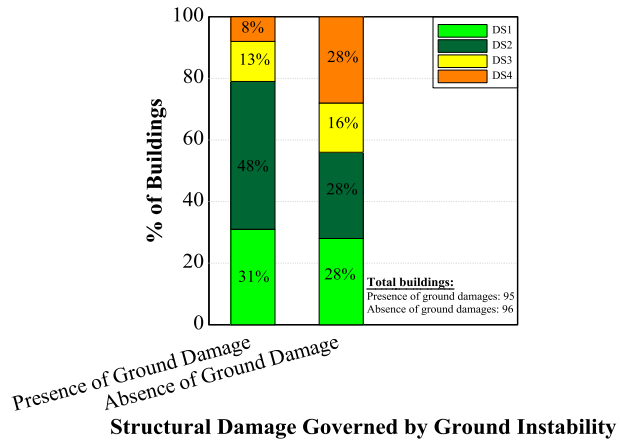


Fig. 10 The influence of ground instability on structural damage



damage (DS3–DS4). In contrast, there were 96 out of 191 RCFMI buildings that sustained earthquake-induced shaking alone without being affected by ground instability, with 42 out of 96 (44%) buildings experiencing moderate to heavy damage (DS3–DS4). The results suggest that ground instability had no discernible influence on observed damage to RCFMI buildings in the Canterbury earthquakes.

Following the 2011 Lyttelton earthquake it was observed that taller RCFMI buildings sustained a higher degree of damage in comparison with low-rise RCFMI buildings (see Fig. 11). The majority (more than 80%) of RCFMI buildings having 3+ stories sustained moderate to heavy damage (D3–D4), whilst most of the RCFMI buildings having 1–2 stories experienced less severe damage.

Figure 12 presents the recorded level of damage sustained by RCFMI buildings following the 2011 Lyttelton earthquake, correlated against the period of construction. It was observed that RCFMI buildings constructed between the 1910s and the 1930s (more than 45% of the total number of surveyed buildings) sustained moderate to heavy damage

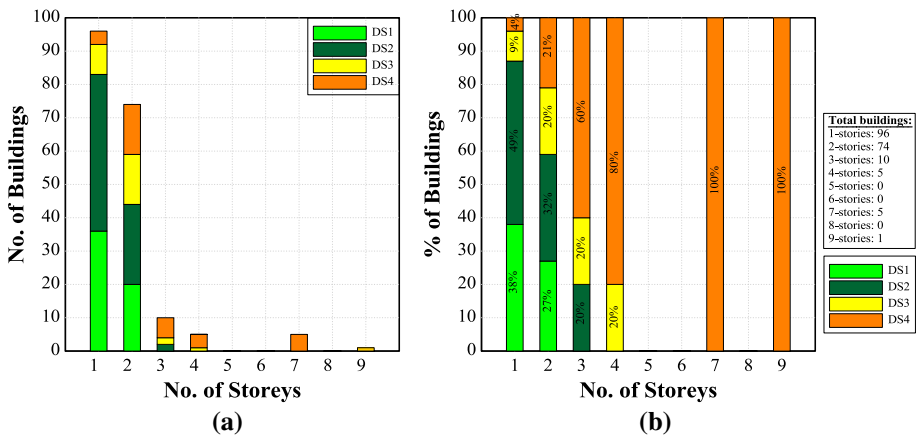


Fig. 11 The influence of building height on damage to RCFMI buildings. **a** Total number of buildings, **b** percentage of buildings

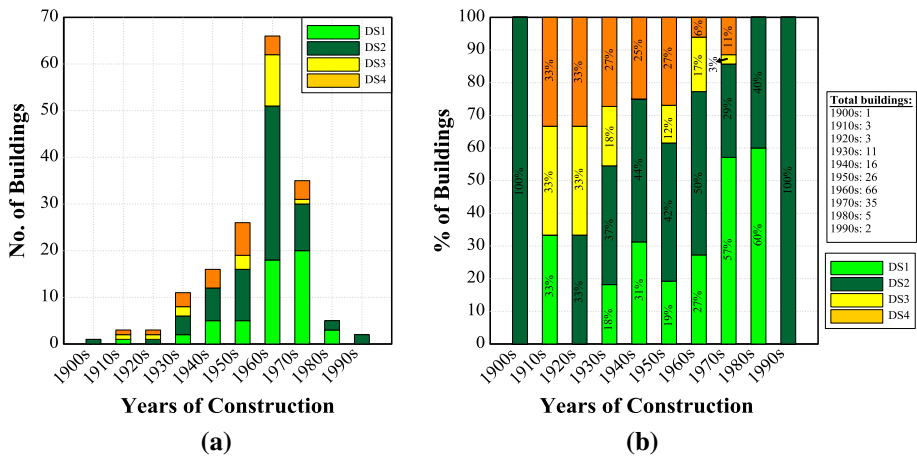


Fig. 12 The influence of period of construction on building damage. **a** Total number of buildings, **b** percentage of buildings

(DS3–DS4). It is noted that the first New Zealand seismic code were introduced in 1935 (NZS 95 1935). Therefore, the observed earthquake vulnerability of RCFMI buildings constructed from the 1910s to the 1930s is not surprising.

As opposed to RCFMI buildings constructed before the establishment of the New Zealand Seismic Code in 1935, RCFMI buildings constructed from the 1940s onwards exhibited better performance, which again is not surprising because seismic requirements would have been incorporated into the design process. In 1976 the New Zealand seismic code had a significant elevation in seismic criteria (NZS 4203 1976) which required seismic detailing and ductility for RC frames, and also required that masonry infill walls be tied to floor diaphragms (Beattie et al. 2008). Therefore, it is predictable that RCFMI buildings constructed from the late 1970s onwards performed satisfactorily during the earthquakes.

6 Damage Probability Matrix (DPM)

A Damage Probability Matrix (DPM) is commonly utilised to illustrate the level of building damage due to ground motion intensities arising from earthquakes. The ground motion data in the September 2010 and February 2011 earthquakes were accurately recorded by the GeoNet strong motion recorders (Lin et al. 2016), with the conditional PGA contours estimated using a Ground Motion Prediction Equation (GMPE) procedure as described by Bradley and Hughes (2012). In the study reported herein the estimated PGA for each building was correlated against the damages states presented in Fig. 7, with the associated Damage Probability Matrix (DPM) for the RCFMI buildings in the 2011 Lyttelton earthquake presented in Table 3. In addition, the distribution and damage states of the RCFMI buildings, and the corresponding PGA levels shown as contours, are presented in Fig. 13.

As seen in Fig. 13, during the 2011 Lyttelton earthquake the RCFMI buildings in the Christchurch region experienced PGA levels ranging from 0.20 to 1.00 g. These high PGA levels were a result of the epicentre occurring at a shallow depth and being located close to the RCFMI building population. The majority of RCFMI buildings (177 out of 191; 93%) experienced PGA levels ranging from 0.30 to 0.59 g.

Table 3 Damage Probability Matrix (DPM) correlating building damage to peak ground acceleration (PGA) for the 2011 Lyttelton earthquake

Damage states	Peak ground acceleration (g)										Total
	0.10–0.19	0.20–0.29	0.30–0.39	0.40–0.49	0.50–0.59	0.60–0.69	0.70–0.79	0.80–0.89	0.90–1.00		
DS1	5	16	26	8	1						56
DS2	7	14	34	17					1		73
DS3		13	13	1							27
DS4		6	29								35
Total	12	49	102	26	1				1		191



Fig. 13 The distribution of RCFMI buildings with different damage states that were subjected to varying PGA levels in the 2011 Lyttelton earthquake. Light Green, DS1; Dark Green, DS2; Yellow, DS3; Orange, DS4

It was also observed that some RCFMI buildings that were clustered together in the Christchurch CBD were assigned heavy damage (DS4) and were subjected to PGA levels ranging from 0.43 to 0.49 g. The high level of damage that occurred to these RCFMI buildings in the Christchurch CBD was attributed to their date of construction, recognising that the city developed outwards from the city centre and therefore the buildings located in the CBD were generally the oldest and consequently the most seismically vulnerable (see Fig. 14). Unfortunately, in most cases it was not possible to obtain accurate records for the specific ages of these buildings. According to photographic records obtained from Christchurch City Libraries, these buildings were well established in the 1950s (see Fig. 14b) and the actual date of construction of these buildings is therefore surmised to be prior to the 1950s. Therefore, the observed seismic vulnerability of these pre-1950s buildings was unsurprising.

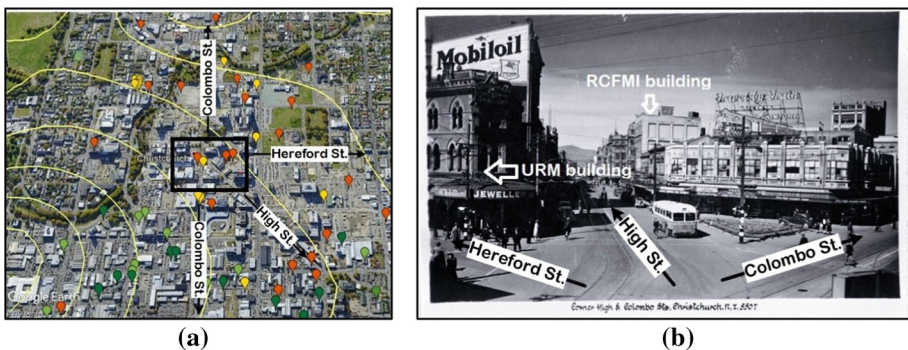
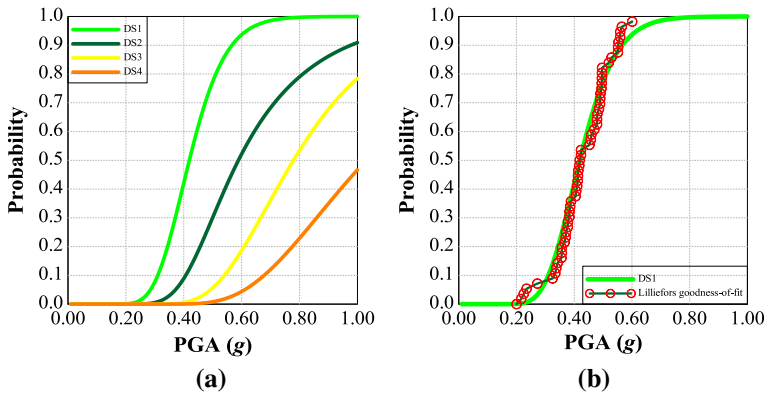


Fig. 14 ‘Triangle corner’ at the intersection of Hereford St., High St., and Colombo St., illustrating early commercial buildings having different construction forms. **a** Map view of ‘Triangle corner’, **b** photograph from the 1950s (Reproduced from Christchurch City Libraries)

Table 4 Estimated fragility functions based on available post-earthquake data following the 2011 Lyttelton earthquake

Damage states (DS)	Median (θ)	SD (β)
DS1	0.419	0.235
DS2	0.564	0.429
DS3	0.682	0.350
DS4	0.430	0.531

**Fig. 15** Empirical fragility curves and Lilliefors goodness-of-fit test for RCFMI buildings following the 2011 Lyttelton earthquake. **a** Empirical fragility curves, **b** Lilliefors goodness-of-fit test for the resulting fragility curve at DS1

7 Empirical fragility curves

The Lognormal Cumulative Distribution method was utilised to derive empirical fragility curves because this method is generally used to represent the fragility relationship (Rota et al. 2008; Kappos et al. 2006; Sarabandi et al. 2004). The fragility relationship can be defined from the occurrence probability of a certain damage state as a function of PGA as follows:

$$P(\text{DS} \geq \text{DS}_i | \text{PGA}) = \Phi \left(\frac{\ln(\text{PGA} / \theta_{\text{DS}_i})}{\beta_{\text{DS}_i}} \right) \quad (1)$$

where: Φ , is the standard normal cumulative distribution function; θ_{DS_i} , is the expected median value of PGA when buildings reach a certain damage state; β_{DS_i} , is the natural logarithm standard deviation of PGA for different damage states.

The empirical fragility curve for each damage state was generated using the corresponding value of median (θ_{DS_i}) and standard deviation (β_{DS_i}), which were calculated based on the Maximum Likelihood Estimation (MLE) method. The calculated median and standard deviation values are presented in Table 4, with the corresponding empirical fragility curves generated for each damage state as presented in Fig. 15a. It is considered that the resulting standard deviations presented in Table 4 were low because the number of damaged RCFMI

buildings that were surveyed by reconnaissance teams on behalf of CCC and CERA following the 2011 Lyttelton earthquake was small (Table 3). A large number of additional undamaged RCFMI buildings were observed during the sidewalk survey conducted in October 2016, but these undamaged RCFMI buildings were not included into the post-earthquake database. It is thought that the RCFMI buildings that were undamaged during the 2011 Lyttelton earthquake were deemed to not require further seismic assessment and were considered to be safe to occupy. Consequently the post-earthquake database for RCFMI buildings that was compiled following the 2011 Lyttelton earthquake has a small number of entries. However, the quality of the database is considered to be reliable because the damaged RCFMI buildings were assessed by reconnaissance teams employing a Level 2 detailed seismic assessment procedure. According to Rossetto et al. (2013), a detailed engineering survey for a specific class of building, such as RCFMI buildings, that is conducted by reconnaissance teams following an earthquake can generate a sample size that may range from small to large, but with the survey finding ranking highly for the reliability of the observations. Therefore, the calculated fragility functions such as median and standard deviation used to generate fragility curves employing the database collected from the 2011 Lyttelton earthquake were deemed to be trustworthy.

The quality of fit between the empirical fragility curves and the observation data was assessed using the Lilliefors goodness-of-fit test. The proposed fragility curves were tested at the 5% significance level utilising the Lilliefors goodness-of-fit test, with these curves deemed to be acceptable if passing the 5% significance level, thereby indicating that the proposed fragility curves fitted the observation data with a 95% level of acceptance. An example of the Lilliefors goodness-of-fit test for RCFMI buildings assigned to DS1 is illustrated in Fig. 15b.

In several previous studies conducted in European countries, fragility curves have been generated based on available post-earthquake datasets. For example, the study conducted by Rossetto and Elnashai (2003) considered 19 earthquake events, including both European and non-European earthquakes, and led to the derivation of fragility curves based on the observed earthquake response of a total of 340,000 buildings, with approximately 2720 buildings comprised of a RCFMI construction system. The available building datasets were processed by equal weighting statistics and nonlinear regression methods. The observed extents of damage for all buildings with different construction systems were classified as Slight, Light, Moderate, Extensive, Partial Collapse and Collapse, with the level of damage for RCFMI buildings being assigned to Slight (DS1), Light (DS2), Moderate (DS3) and Extensive (DS4) damage. The resulting fragility curves were generated by adopting the cumulative beta distribution method, with the median and standard deviation calculated based on nonlinear regression of observational statistical data.

In a similar manner as previously undertaken by Rossetto and Elnashai (2003), a study by Del Gaudio et al. (2016a) utilised nonlinear regression analysis to process the post-earthquake performance observations for 5033 RCFMI buildings considered from a total of 7597 surveyed RC buildings subjected to the 2009 L'Aquila, Italy earthquake. The level of damage was categorised as Slight (DS1), Light (DS2), Moderate (DS3), and Extensive (DS4) damage states in accordance with EMS-98 (Grünthal 1998). The fragility curves for these buildings were derived by adopting the Lognormal Cumulative Distribution method, with the median and standard deviation being calculated by employing the weighted Least Square Estimation and the Maximum Likelihood Estimation (MLE) methods.

The empirical fragility curves derived from the 2011 Lyttelton earthquake were subsequently compared with previous empirical fragility curves for RCFMI buildings that were generated by Rossetto and Elnashai (2003) and by Del Gaudio et al. (2016a) (see Fig. 16).

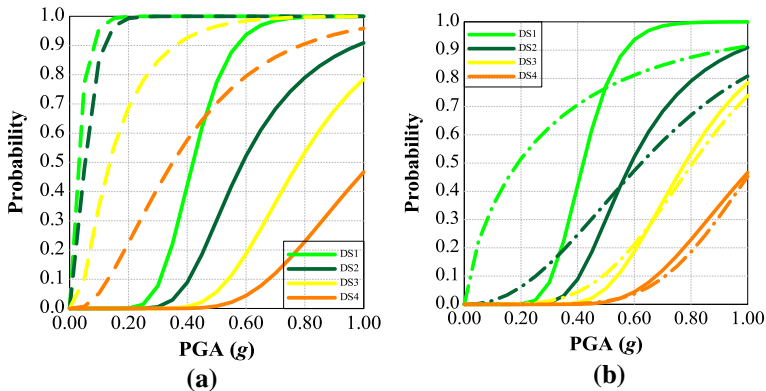


Fig. 16 Comparison of proposed empirical fragility curves with previously reported fragility curves. **a** Rossetto and Elnashai (2003), **b** Del Gaudio et al. (2016a)

As shown in Fig. 16a, the fragility curves generated by Rossetto and Elnashai (2003) (represented by dashed lines) for each damage state indicated significantly higher expected levels of damage for a prescribed PGA level when compared to the proposed fragility curves generated in the Christchurch study (illustrated in solid line). It is recognised that the fragility curves developed by Rossetto and Elnashai (2003) utilised a large post-earthquake dataset from 19 earthquake events in different countries, with no specific classification for buildings having different attributes. Consequently, the broad scope of building typologies and earthquake characteristics is likely to have led to large variabilities associated with the derived empirical relationships between ground motion data and building damage observed in the earthquakes, thereby resulting in a higher seismic fragility when compared to RCFMI buildings in the Christchurch region during the 2011 Lyttelton earthquake.

As opposed to the fragility curves derived by Rossetto and Elnashai (2003) utilising post-earthquake datasets from various earthquake events around the world, the fragility curves generated by Del Gaudio et al. (2016a) were derived from the 2009 L'Aquila, Italy earthquake. As seen in Fig. 16b, RCFMI buildings assigned to damage state DS1 (Slight) in the 2011 Lyttelton earthquake had a lower probability of occurrence when compared to RCFMI buildings assigned to damage state DS1 (Slight) that were damaged in the 2009 L'Aquila earthquake. In contrast, in the 2009 L'Aquila earthquake and the 2011 Lyttelton earthquake the damage to RCFMI buildings had a relatively similar probability of occurrence for the Light (DS2), Moderate (DS3) and Extensive (DS4) damage states. It is theorised that the discrepancies between the fragility curves generated from the 2009 L'Aquila earthquake and the 2011 Lyttelton earthquake are associated with the different definitions adopted for the level of damage determination. The study by Del Gaudio et al. (2016a) explicitly considered the extent of damage to infill partitions and vertical structures in order to determine the damage state, whereas the level of damage to RCFMI buildings in the 2011 Lyttelton earthquake was influenced by the usability category that had been assigned in the Level 2 detailed seismic assessments. The seismic fragility functions derived from these two studies are also governed by the significant differences associated with the respective nature of the RCFMI buildings in the two countries of Italy and New Zealand, and the specific characteristics of the 2009 L'Aquila earthquake and the 2011 Lyttelton earthquake, resulting in differences in RCFMI building response during the two earthquakes.

Empirical building damage data collected following large earthquakes and the resulting fragility curves generated from these damage observations can be used to predict damage scenarios for existing buildings in future earthquakes, as has previously been undertaken by researchers from European countries such as in Italy and Greece (Dolce et al. 2003; Rossetto and Elnashai 2003; Rota et al. 2008; Del Gaudio et al. 2016b). The study conducted by Dolce et al. (2003) evaluated the vulnerability of buildings subjected to earthquakes by combining the building typology analysis with expert judgement. The probability distribution of building damage was determined, and the buildings were assigned into vulnerability classes. Conversely, the study undertaken by Kappos et al. (2006) assessed the building vulnerability by combining the available statistical data from post-earthquake data collection with nonlinear static and dynamic structural analysis for the specific buildings representing each vulnerability class. The study by Dolce et al. (2006) presented a comparison between the method used in Italy and the method used in Greece, with these two methods applied to forecast damage scenarios for existing buildings in Potenza, Italy.

The accuracy of empirical fragility curves is heavily reliant upon the quality of the post-earthquake data acquired, which is influenced by the survey method, the definition of building damage states, and the estimation of ground motion intensity level. It is recognised that the level of damage for some RCFMI buildings was not reported in the CEBA database, which introduced some uncertainty regarding the accuracy of the developed fragility relationships. Despite these uncertainties, the proposed empirical fragility curves from the 2011 Lyttelton earthquake are expected to provide a reliable insight regarding the seismic vulnerability of commercial RCFMI buildings in New Zealand because it is assumed that the primary characteristics of RCFMI buildings is relatively similar across the country.

It is highlighted that the available dataset of RCFMI buildings damaged during the 2010/2011 Canterbury earthquakes can be employed for earthquake damage scenarios for existing RCFMI buildings on a national scale. The typological building damage collected from the Canterbury earthquakes can be categorised into vulnerability classes, and the expected damage for the existing building stocks of different regions can be forecasted by utilising earthquake ground motion scenarios appropriate to each region. Because the study reported herein has not included such seismic vulnerability assessments and earthquake damage scenarios, these topics will be addressed in a future study.

8 Conclusions

Available earthquake performance data from 191 RCFMI buildings was assessed following the 2010/2011 Canterbury earthquakes. It was observed that the majority of RCFMI buildings revealed an adequate performance during the earthquakes, with a considerable number of these buildings assigned green placards (68%). In addition, their current state showed that a comparatively small number of RCFMI buildings had been demolished (26%), while the majority of these buildings remained standing (74%).

Observations revealed that RCFMI buildings with clay brick infill sustained more damage when compared to FCFMI buildings infilled by concrete block, but, there was no clear evidence that wall morphology (solid or cavity-wall construction) exhibited a significant effect on building performance in the Canterbury earthquakes. In addition, there was little evidence that the characteristics of the footprint geometry significantly influenced the seismic performance of RCFMI buildings in the Canterbury earthquakes, primarily because there were a relatively low number of RCFMI buildings having irregular footprints that

sustained moderate to heavy damage (D3–D4). Additionally, a small proportion of RCFMI buildings exhibited moderate to heavy damage (DS3–DS4) induced by ground instability during the Canterbury earthquakes.

It was reported that with increasing building height, damage levels were shown to increase and a large number of RCFMI buildings having 3+ stories experienced moderate to heavy damage (DS3–DS4). RCFMI buildings constructed during the period between the 1910s and the 1930s were more likely to be earthquake-vulnerable because it was shown that many buildings from this era had a high level of damage compared to more modern buildings.

During the earthquakes, most RCFMI buildings were subjected to PGA values of between 0.30 and 0.59 g. Empirical fragility curves adopting the lognormal distribution method were generated, and the fragility functions were estimated based on the Maximum Likelihood Estimation (MLE) method. The proposed empirical fragility curves presented herein were compared to previous empirical fragility curves developed in different countries. The comparison revealed that the empirical fragility curves varied, with this variation attributed to the different characteristics of the damaging earthquakes and variations in the characteristics of RCFMI buildings in each country. Despite empirical data having a large uncertainty due to the reliability of post-earthquake data acquisition procedures, the empirical fragility curves proposed herein are expected to provide a reliable insight regarding the seismic vulnerability of RCFMI buildings in New Zealand.

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