ENG.UOA.0002.1

# Addendum Report to the Royal Commission of Inquiry


# The Performance of Earthquake Strengthened URM Buildings in the Christchurch CBD in the 22 February 2011 Earthquake

by

Associate Professor Jason M. Ingham

The University of Auckland

and

Professor Michael C. Griffith

The University of Adelaide

October, 2011

# Acknowledgements

This report has drawn extensively upon the works of a number of current and former postgraduate students, whose involvement is acknowledged below:

- Dmytro Dizhur is a doctoral candidate at the University of Auckland. Dmytro has expertise in the field testing of unreinforced masonry buildings, and has played a significant role in collecting and analysing the data reported throughout this report. Many of the images presented in this report were provided by Dmytro, whose doctoral studies are supported financially by a University of Auckland Doctoral Scholarship.
- Lisa Moon is a doctoral candidate at the University of Adelaide. Lisa's doctorate will report on the damage to unreinforced clay brick buildings in the February 2011 earthquake, and Lisa made major contributions to this report. Lisa's doctoral studies are supported financially by a University of Adelaide Doctoral Scholarship and her expenses in Christchurch were supported by the New Zealand Natural Hazards Research Platform.
- Ronald Lumantarna is a doctoral candidate at the University of Auckland. Ronald's doctoral studies are focussed on developing a knowledge base regarding the material properties of New Zealand's unreinforced masonry buildings, for use by practicing structural engineers in their seismic assessment and retrofit designs. Ronald's doctoral studies are supported financially by a University of Auckland Doctoral Scholarship.

Expertise associate with the seismic response of unreinforced masonry buildings was developed at the University of Auckland as part of the 'Retrofit Solutions' research project that was funded by the New Zealand Foundation for Research Science and Technology grant UOAX0411 during the period 1 July 2004 to 30 September 2010. The website associate with this project may be accessed at www.retrofitsolutions.org.nz

Expertise associated with the seismic response of unreinforced masonry buildings was developed at the University of Adelaide, in part, through research funded by the Australian Research Council over the period 1 January 2004 to 31 December 2010.

Data on the performance of unreinforced masonry buildings in the 22 February 2011 earthquake was collected as part of the Natural Hazards Research Platform Recovery Project 'Project Masonry'. The Natural Hazards Research Platform maintains a website at http://www.naturalhazards.org.nz/.

The authors wish to thank the numerous professional structural engineers and building owners who have provided valuable data, opinions, expertise and their experiences. In particular: John Hare, Stuart Oliver and others from Holmes Consulting Group Ltd.; Paul Campbell, Will Parker and others from Opus International Consultants Ltd.; Win Clark, Cecil DelaRue, Fiona Wykes and others from the Civil Defence Heritage team; Hossein Derakhashan and others from Aurecon New Zealand Ltd.; Andrew Marriot and others from Marriot Consulting Engineers Ltd.; and URS Consulting Ltd.

The authors thank Ronald Lumantarna for conducting and providing mortar and clay brick compression strength results. EQ STRUC Ltd. is thanked for providing expertise and test equipment and Hilti (NZ) Ltd, Reids Construction Systems Ltd and Sika (NZ) Ltd is thanked for proving materials in order to conduct in-field anchor pull-out tests.

Darryl and Alistair from the Civil Defence demolition team, Graceworks Demolition, Ward Demolition, Southern Demolition, Nikau Demolition, and other demolition companies are thanked for allowing site investigation and access for sample collection during demolition of damaged buildings.

Stephanie German and Jazalyn Dukes from Georgia Institute of Technology and Chaminda Konthesingha from the University of Newcastle are also thanked for their assistance and contributions.

The authors acknowledge the financial support for Project Masonry from the New Zealand Natural Hazards Research Platform. The testing of adhesive anchors was undertaken in conjunction with the RAPID grant CMMI-1138614 from the US National Science Foundation.

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# Glossary and abbreviations

| | |
|---|---|
| Acceleration response spectra | A diagram that shows the peak ground acceleration that a building of a specific period will be subjected to. The spectra can be used to assess both the seismic inertial forces induced in an elastically responding structure and the amount of induced displacement relative to the ground |
| Anchor | A structural element used to assist with connecting various parts of the building, such as between floors and walls. In URM buildings anchors have frequently been added after the original construction |
| Braced frame | A structural system that consist of beams, columns, and stiff diagonal braces that perform like shear walls, but use less material (see also 'moment frame'). In masonry buildings a braced frame would normally be constructed of structural steel |
| Cantilever | A structural element that is rigidly connected at one end, and is unsupported along its length and at its other end |
| Cavity | A method of wall construction where there is an inner and an outer leaf (or layer) of masonry and a central gap (cavity) that has the function of providing ventilation and a pathway for moisture to exit the wall (see also solid construction). The air in the cavity also has insulation value |
| Central Business District (CBD) | The terms of reference for the Royal Commission define the Christchurch City CBD to mean the area bounded by the 4 avenues (Bealey Avenue, Fitzgerald Avenue, Moorhouse Avenue, and Deans Avenue); and Harper Avenue |
| Diaphragm | A horizontal or inclined structural element within a building that has the function of providing stiffness and stability to perpendicular walls and to transmit loads to these walls. In unreinforced masonry buildings this term is normally applied to mid-height floors and to roofs, which in both cases are usually constructed of timber |
| Ductility | The ability of a building or a structural element of a building to be able to plastically deform without losing strength |

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| Earthquake Prone Building | A building having an expected earthquake performance that is less than or equal to 33% of that of an equivalent new building correctly designed to current standards and located at the same site (see also %NBS below) |
| Earthquake Risk Building | A building having an expected earthquake performance that is between 34% and 67% of that of an equivalent new building correctly designed to current standards and located at the same site (see also %NBS below) |
| Facade | The front of a building, that typically has a more pleasant appearance than the sides and rear of the building |
| Fibre Reinforced Polymer (FRP) | A high strength lightweight material composed of synthetic fibres held within a polymer layer than can be used to improve the earthquake performance of a building |
| Gable wall | The vertical triangular section at the top of a wall, located between the two sloping ends of a pitched roof |
| Iconic buildings | Historically or culturally significant buildings |
| Importance Level | The importance of a building in and after an earthquake. Buildings that are expected to contain large numbers of people or buildings that are expected to have an emergency function after an earthquake have higher importance. |
| In-plane behaviour | Behaviour that occurs in the direction parallel to the orientation of the structural element, which is typically a wall. The term is often used to describe failure, where for instance door and window openings in a wall may no longer have right angle corners (see also out-of-plane behaviour) |
| Intensity | A measure of the effect of an earthquake at a particular site, often measured in terms of the maximum ground acceleration at that location |
| Leaf (also referred to as wythe) | A leaf of masonry refers to a single vertical layer of masonry bricks. In most unreinforced masonry construction it is common to have a multi-leaf wall that has a width corresponding to the thickness of several bricks. In North America the term 'wythe' is usually used instead. |
| Magnitude (M) | A measure of the total energy released by the earthquake, originally based upon the Richter Scale but now determined using a revised technique called Moment Magnitude |
| Mega Pascal (MPa) | A unit of stress typically used to describe the strength of materials used in structural engineering. The same units are also used to describe material stiffness |
| Moment frame | A structural system that consist of beams and columns that are connected at moment-resisting joints (see also 'braced frame'). In masonry buildings a moment frame would normally be constructed of either structural steel or reinforced concrete |
| Near Surface Mounting (NSM) | An earthquake strengthening technique where slots are cut into a masonry wall and strengthening elements are inserted into the slots. The reinforcing element can then be covered over such that it is located near the surface rather than on the surface of the wall |
| Out-of-plane behaviour | Behaviour that occurs in the direction perpendicular to the orientation of the structural element, which is typically a |

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| | wall. The term is often used to describe failure, where for instance a wall may deform outwards or completely collapse into the adjacent street or alley (see also in-plane behaviour) |
| Parapet | A section of wall that typically extends above the intersection between the roof and wall. The parapet may be either triangular or rectangular and is an ornamental element that may report the name or construction date of the building |
| Period | A structural characteristic that is used to describe how the building will oscillate in an earthquake. The period is measured in seconds and is dependent on a building's mass and its stiffness. The term describes the time taken for a building to complete one full cycle of lateral deformation |
| Pier | A structural element oriented vertically and typically bounded by window or door openings. The term is analogous to 'wall' but refers to a part of a wall when the wall has openings. |
| Render | A layer of mortar that is applied to the exterior of a masonry building |
| Report No. 1 | The authors' first report to the Royal Commission of Inquiry, having the following bibliographic details: Ingham, J. M. & Griffith, M. C. (2011). 'The Performance of Unreinforced Masonry Buildings in the 2010/2011 Canterbury Earthquake Swarm', Report to the Royal Commission of Inquiry into Building Failure Caused by the Canterbury Earthquakes', Accessed at: http://canterbury.royalcommission.govt.nz/Technical-Report---The-Performance-of-Unreinforced-Masonry-Buildings-in-the-2010-2011-Canterbury-Earthquake-Swarm |
| Sarking boards' | The use of wood panels under the shingles of a roof to provide support |
| Securing | The term 'securing' is used here to refer to temporary securing measures such as the addition of anchors to connect wall elevations to roof and floor diaphragms for temporary securing, or the addition of steel straps to hold cracked building corners together. |
| Seismic zone factor | A factor that numerically describes the seismicity of a region |
| Shear wall | A wall that has the structural function of transmitting lateral loads to the foundations, and frequently has few or no openings (see also 'in-plane behaviour') |
| Shoring | Refers to the addition of bracing members (usually braced to the ground or an adjacent building) to stabilise building elements from damage in subsequent aftershocks. |
| Shotcrete | Concrete that is pumped through a hose and sprayed on to the surface of a wall |
| Solid construction | Wall construction where multiple leafs (or layers) of masonry are used to create the wall thickness, without including a cavity |
| Spandrel | A structural element oriented horizontally and typically bounded by window or door openings. The term is analogous to 'wall' but refers to a part of a wall when the wall has openings. |

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| Strongback | A structural element that is added to a wall to increase its stiffness, typically oriented vertically |
| Typology | A systematic classification of types that have similar characteristics |
| Unreinforced masonry (URM) | Construction of clay brick or natural stone units bound together using lime or cement mortar, without any reinforcing elements such as steel reinforcing bars. The term herein is used for a class of building that in North America is referred to as a 'URM bearing wall building' and furthermore is herein used to exclude reference to masonry infilled concrete frames and to confined masonry construction. |
| Territorial Authorities | Territorial authorities are the second tier of local government in New Zealand, below regional councils, and are based on community of interest and road access. There are 67 territorial authorities |
| Veneer | Masonry veneer refers to the practice of using masonry to form the external vertical envelope of a building when the internal structure of the building may be of a different form. In New Zealand it is common to construct timber framed residences having masonry veneer. The term may also be used to describe a decorative outer leaf of brick in multi-leaf construction |
| %NBS | Percentage New Building Standard: A number that scores the expected earthquake performance of a building compared to that of an equivalent new building correctly designed to current standards and located at the same site |

# Executive Summary

Following the 22 February 2011 Christchurch earthquake a comprehensive damage survey of the unreinforced masonry (URM) building stock of Christchurch was undertaken, principally led by Lisa Moon, a doctoral research student from the University of Adelaide, and by Dmytro Dizhur, a doctoral research student from the University of Auckland. Lisa and Dmytro were supported by a number of research colleagues and the information contained in this report is attributable to their collective efforts.

In late July 2011 the authors provided to the Royal Commission of Inquiry a report that provided background information on the characteristics of the national URM building stock and details of the damage information to Christchurch URM buildings that had been collated following the 4 September 2010 Darfield earthquake. In September 2011 the authors were requested to prepare an Addendum Report documenting the damage to Christchurch URM buildings in the 22 February 2010 earthquake. Because of the wealth of data that has been collected and the need to expeditiously provide the Commission with this Addendum Report, a decision was made to exclusively focus on URM buildings located in the CBD zone. The collected damage data was supplemented by technical drawings and calculations on file with the Christchurch City Council, plus the use of post-earthquake aerial photography where available. The procedure used to collect and process information associated with damage, general analysis and interpretation of the available survey data for 370 buildings, the performance of earthquake strengthening techniques, the influence of earthquake strengthening levels on observed damage, and finally material and in-situ testing are reported here.

With respect to URM buildings in the 22 February 2011 Christchurch earthquake the principal findings from this subsequent investigation are:

- 97% of URM buildings that had received no prior earthquake strengthening were either seriously damaged (i.e. suffered heavy or major damage) or collapsed. 90%

of unretrofitted buildings have since been demolished or are currently scheduled to be demolished

- 63% of all URM buildings in the CBD had received some form of earthquake strengthening
  - 68% of heritage and protected URM buildings had received some form of earthquake strengthening, whereas 58% of buildings not listed as heritage or protected had received some form of earthquake strengthening.

With respect to URM buildings that had received some form of earthquake strengthening:

- Of those URM buildings that had been earthquake strengthened to less than 33%NBS, 60% were seriously damaged (i.e., suffered heavy or major damage) or collapsed
- Of those URM buildings that had been earthquake strengthened to 34-67%NBS, 72% were seriously damaged or collapsed
- Of those URM buildings that had been earthquake strengthened to 67-100%NBS, only 24% were seriously damaged or collapsed
- Of those URM buildings that had been earthquake strengthened to 100%NBS or greater, none were seriously damaged or collapsed.

With respect to earthquake strengthening methods:

- 44% of restrained parapets failed, compared with failure of 84% of unrestrained parapets. Whilst parapet restraint generally improved earthquake performance, it is clear that many parapet restraints failed to perform as intended. Clearly, parapet retrofits provided some earthquake resistance but probably less would be expected. This somewhat surprising result was partly attributable to the observed poor performance of adhesive anchorage systems and may have also been due to parapets being secured to roof systems having diaphragms that were too flexible
- 57% of restrained gable end walls failed, compared with 88% of unrestrained gable end walls. Similar to parapets, whilst gable restraint also generally improved earthquake performance, it is clear that many gable restraints also failed to perform as intended.

Further investigation should be conducted to ascertain which parapet and gable wall retrofit techniques were most effective and why, in order to improve the effectiveness and reliability of earthquake strengthening solutions in the future.

There is clear evidence that installed earthquake strengthening techniques reduced damage levels, that Type A+B retrofits were significantly more effective at reducing overall structural damage, and that shotcrete strengthened wall retrofits and added cross wall retrofits appeared to be more effective than steel strongback retrofits, again probably due to better material deformation compatibility with masonry.

Additional findings are that:

- In general, the earthquake strengthening techniques applied to Christchurch URM buildings are consistent with the earthquake strengthening techniques used in California. California is specifically used as a benchmark because of the repeated number of large earthquakes that have been experienced there in the previous several decades, and the similarity in the characteristics of its URM building stock. Also, the authors' first report was peer reviewed by two people resident in California, and the authors encourage all readers to incorporate those peer review reports into the collective information available on the earthquake performance of URM buildings
- Observations indicate that the characteristics of the Christchurch URM building stock located outside the CBD zonation differ somewhat from that reported here, with a greater proportion of URM buildings located outside the CBD having cavity wall construction. This data will need to be amalgamated with the data reported herein in order to obtain a complete understanding of the overall damage to Christchurch URM buildings.

# Section 1:

# Introduction

This section provides introductory information on the scope and purpose of this report.

This report is an addendum to the authors' earlier report to the Royal Commission of Inquiry into Building Failure Caused by the Canterbury Earthquakes[1], hereafter referred to as 'Report No. 1'. The Commission specifically requested supplementary information on the performance of unreinforced masonry (URM) buildings in the Christchurch Central Business District (CBD, as defined in the Terms of Reference for the Royal Commission), for the earthquakes that occurred on 22 February 2011 and later. In particular, the Commission specifically requested further details on the performance of URM buildings located in the CBD that had received prior earthquake strengthening[2].

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[1] Ingham, J. M. & Griffith, M. C. (2011). 'The Performance of Unreinforced Masonry Buildings in the 2010/2011 Canterbury Earthquake Swarm', Report to the Royal Commission of Inquiry into Building Failure Caused by the Canterbury Earthquakes', Accessed at: http://canterbury.royalcommission.govt.nz/Technical-Report---The-Performance-of-Unreinforced-Masonry-Buildings-in-the-2010-2011-Canterbury-Earthquake-Swarm

[2] Note that the term 'Earthquake Strengthening' is used throughout this report to refer to the exercise of improving the seismic performance of a building in an earthquake. For unreinforced masonry buildings this improvement is almost always associated with the incorporation of additional structural components or elements that do increase the building's strength. However, in the more general sense the phrase can be applied to alternative techniques also, which may modify the building's structural dynamic characteristics in a manner that does not make the building stronger, but does allow the building to perform better when subjected to earthquake loads.

In response to this request the authors and their postgraduate student research team compiled a database on the performance of 370 URM buildings located in the Christchurch CBD, and accessed technical drawings from Christchurch City Council for 74 buildings that had been earthquake strengthened. This report summarises the collated data from this database. In summary it was determined that 232 (63%) URM buildings in the Christchurch CBD had some form of earthquake strengthening, with 149 (34%) parapets having restraints installed, out of a total of 435 parapets surveyed. From the collected information it was possible to assign a %NBS value of strengthening to 94 URM buildings in the Christchurch CBD.

## 1.1 Peer review comments

Report No. 1 was peer reviewed by Mr Fred Turner of the Californian Seismic Safety Commission[3] and Mr Bret Lizundia of Rutherford & Chekene from San Francisco[4]. The authors are indebted to both Mr Turner and Mr Lizundia for the high quality of their reviews, and encourage all readers to treat these review comments as an integral part of the overall reporting of the performance of URM buildings in the recent Christchurch earthquakes. In particular, the authors wish to re-state a key comment made by Mr Turner:

> **Limitations of Retrofits:** The recommendations should acknowledge, as evidenced from past retrofit performance, that it is neither practical nor feasible to state conclusively that the public can be effectively protected from "all" falling hazards and that "strengthened URM buildings will survive severe earthquake ground motions." Other similar policy documents use qualifying phrases to characterize the limits of performance objectives such as: "risk reduction programmes" (NZSEE 2006); "reduce the risk of life or injury" (IEBC 2009); "decrease the probability of loss of life, but this cannot be prevented" (IEBC 2006); "compliance with this standard does not guarantee such performance" (ASCE 2006); and "reduce damage and needed repairs" (CHBC, 2010). The reason for proposing these clarifications is that the public should be made aware of the practical limitations of seismic retrofits, considering the margins of safety from collapse and parts of buildings falling, particularly in light of the large known variability and uncertainty of ground motions, as well as variations and uncertainty in the quality of building materials, the states of repair, and the integrity of connections between building components. In a retrofitted URM building, a single masonry unit that may fall from an appreciable height has the potential to be lethal or cause serious injury. Retrofits that represent best practices may not always guarantee that all masonry units will remain in

[3] See http://canterbury.royalcommission.govt.nz/Technical-Report---Peer-Review---The-Performance-of-Unreinforced-Masonry-Buildings-in-the-2010-2011-Canterbury-Earthquake-Swarm

[4] See http://canterbury.royalcommission.govt.nz/documents-by-key/20111003.4/$file/ENG.RUT.0001.pdf

> place, nor that URM buildings will always avoid cost-prohibitive repairs or demolitions after experiencing severe ground motions.

The authors accept the wisdom of this comment, and the uncertainty that it implies. It is also noted that Mr Lizundia has made a similar comment in this regard. It is therefore plausible to theorise that an earthquake strengthened building could perform remarkably well in an earthquake, and that the building may be immediately useable after the earthquake such that the structural engineering objectives of the earthquake strengthening design were achieved, and yet the dislodgement of even one brick could potentially cause a fatality.

### 1.1.1 Structural forms considered in this study

Mr Lizundia has correctly identified that greater clarity is merited regarding the definition of buildings that have been considered in the scope of the current study. The authors have previous research experience with reinforced concrete masonry buildings (see for instance Voon & Ingham (2006) and Voon & Ingham (2007)) and also with both confined masonry construction and buildings constructed using concrete frames with masonry infill, following their post-earthquake research in Sumatra (see Griffith et al. 2010). The authors have also collated damage statistics following the Christchurch earthquake for buildings constructed with a timber frame and having a masonry veneer.

The study reported both herein and in Report No. 1 has exclusively focussed on clay brick unreinforced masonry (URM) buildings, which in North America are referred to as URM bearing wall buildings. This scope was requested by the members of the Royal Commission.

## 1.2 The learning experience provided by the Christchurch earthquakes

It is customary for innovations in structural earthquake engineering to be developed through a process of laboratory experimental testing and supplementary computer modelling, matched with a rational design procedure, such that the structural engineering community discerns the innovation to be appropriate for implementation into actual buildings. In some cases, further in-field testing may be conducted on parts of buildings in which the innovation has been installed, in an attempt to simulate the effect of earthquake loading and identify likely behaviour. Consequently, it is to be expected that many earthquake strengthening techniques are implemented primarily on the basis of laboratory evidence of their suitability, rather than their observed adequate performance in past earthquakes.

Based upon the above, it is suggested that the well documented earthquake performance of such a large number of unreinforced masonry buildings that had received various levels of prior strengthening ranging from unstrengthened (referred to here as 'as-built') to fully strengthened (corresponding to 100%NBS) is a somewhat unique event, particularly when accounting for the number and intensity of the aftershocks that these buildings were subjected to. Consequently, these observations are of major significance in order to gain an updated understanding of the likely seismic performance of previously strengthened URM buildings located not only throughout New Zealand, but also in countries having an analogous stock of URM buildings, such as for instance Australia and West Coast USA. Furthermore, it may be argued that the observations reported herein have relevance to the likely seismic performance of URM buildings worldwide. Mr Lizundia has independently made a similar comment in his peer review.

Since release of Report No. 1 it has been evident that readers and commentators are particularly interested in such aspects as:

- How have buildings performed when they were strengthened to 34%NBS (the minimum level required by the Building Act), to 67%NBS (the minimum level recommended by the New Zealand Society for Earthquake Engineering; NZSEE) and to 100%NBS (corresponding to the seismic loading used for the design of new structures)? The answers to these questions are of tremendous significance as they provide insight into the adequacy of current legislation, and whether alterations to this legislation may be warranted.
- What specific seismic improvement techniques have been shown to perform well when subjected to 'real world' earthquake loading, and what techniques have failed to perform in the manner anticipated. The answers to these questions will likely have significance for future earthquake strengthening strategies employed both in New Zealand and elsewhere worldwide.

## 1.3 Scope and purpose

This document reports the performance of earthquake strengthened unreinforced masonry buildings located in the Christchurch CBD, during the 22nd February 2011 earthquake. The report also provides details on the performance of various earthquake strengthening techniques that had been used in these buildings. The report should be read in conjunction with the authors' Report No. 1 and the peer review reports of Mr Turner and Mr Lizundia.

The Terms of Reference of the Royal Commission of Inquiry are reproduced in Appendix A

## 1.4 Disclaimer

The authors wish to emphasise that they are unqualified to make an informed comment on the cost of earthquake strengthening of URM buildings, beyond the very preliminary comments made in the original report that was primarily a reproduction of the data since made available on the Royal Commission website[5]. No effort is made herein to place a cost on any earthquake strengthening that is reported.

The authors have had no prior involvement with any earthquake strengthening projects in Christchurch that were implemented prior to 22 February 2011. The data reported herein is believed to be correct and reported without bias.

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[5] See http://canterbury.royalcommission.govt.nz/documents-by-key/2011-09-23101/$file/ENG.HOL.0001.pdf

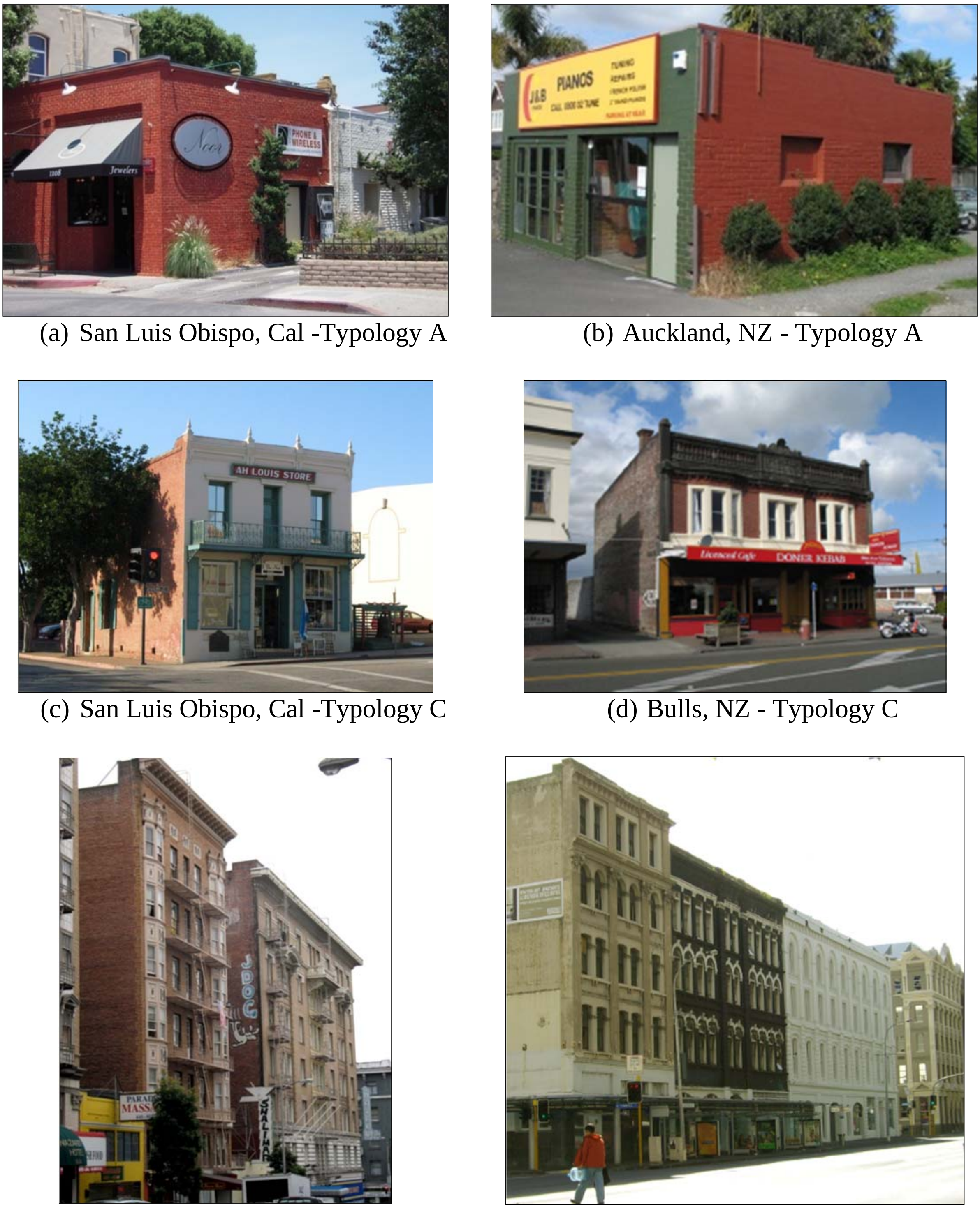
# Section 2:

# URM earthquake strengthening techniques used in California

As noted in Section 1, the authors' Report No. 1 was peer reviewed by Mr Turner and Mr Lizundia, both of California, USA. This section is included herein in an attempt to briefly demonstrate the similarities in earthquake strengthening practices between California and New Zealand, and by extension the suitability of using Californian peer reviewers in this reporting exercise. Furthermore, both reviewers have provided an extensive set of references that have been reproduced herein as sections 9.1 and 9.2. As further outlined below, California has both an analogous stock of URM buildings and a comparable seismicity to New Zealand, but with both more large earthquakes having caused major building damage in California over the last century and with a greater population, it is natural that significant effort relevant to New Zealand has been committed in California to the authoring of references associated with earthquake strengthening of URM buildings. Indeed, many New Zealand engineers have previously consulted US documents when designing earthquake strengthening solutions for New Zealand URM buildings.

## 2.1 Comparison of URM building typologies

Details of the architectural characteristics of the New Zealand URM building stock have been reported in Section 2 of Report No. 1, with representative photographs of different building typologies reported in Figure 2.8 of Report No. 1. Figure 2.1 provides illustrations of the similarities between building typologies in New Zealand and the State of California.




(a) San Luis Obispo, Cal -Typology A

(b) Auckland, NZ - Typology A

(c) San Luis Obispo, Cal -Typology C

(d) Bulls, NZ - Typology C

(e) San Francisco –Typology E

(f) Auckland –Typology F

**Figure 2.1 Comparison of New Zealand and Californian URM building stock**

## 2.2 Seismic improvement approach

The peer review comments of Mr Lizundia provide comprehensive details on standard earthquake strengthening practice is California, with the two documents:

- ASCE 41-06 (also previously known as FEMA 356) titled '*Seismic Rehabilitation of Existing Buildings Standard'*
- FEMA 547 titled '*Guidelines on Seismic Rehabilitation of Existing Buildings*'

being the appropriate documents to use in California for earthquake strengthening of URM buildings. These documents are well known and have been widely consulted by New Zealand structural engineers undertaking earthquake strengthening.

## 2.3 Seismic improvement techniques

### 2.3.1 Parapet restraints

Parapet bracing is typically undertaken using a steel brace angle that is anchored into the top of the parapet and secured back to the roof structure. Typically the roof area where the brace is being connected to also requires localised strengthening to adequately support the brace forces. A typical parapet bracing detail is shown Figure 2.2.

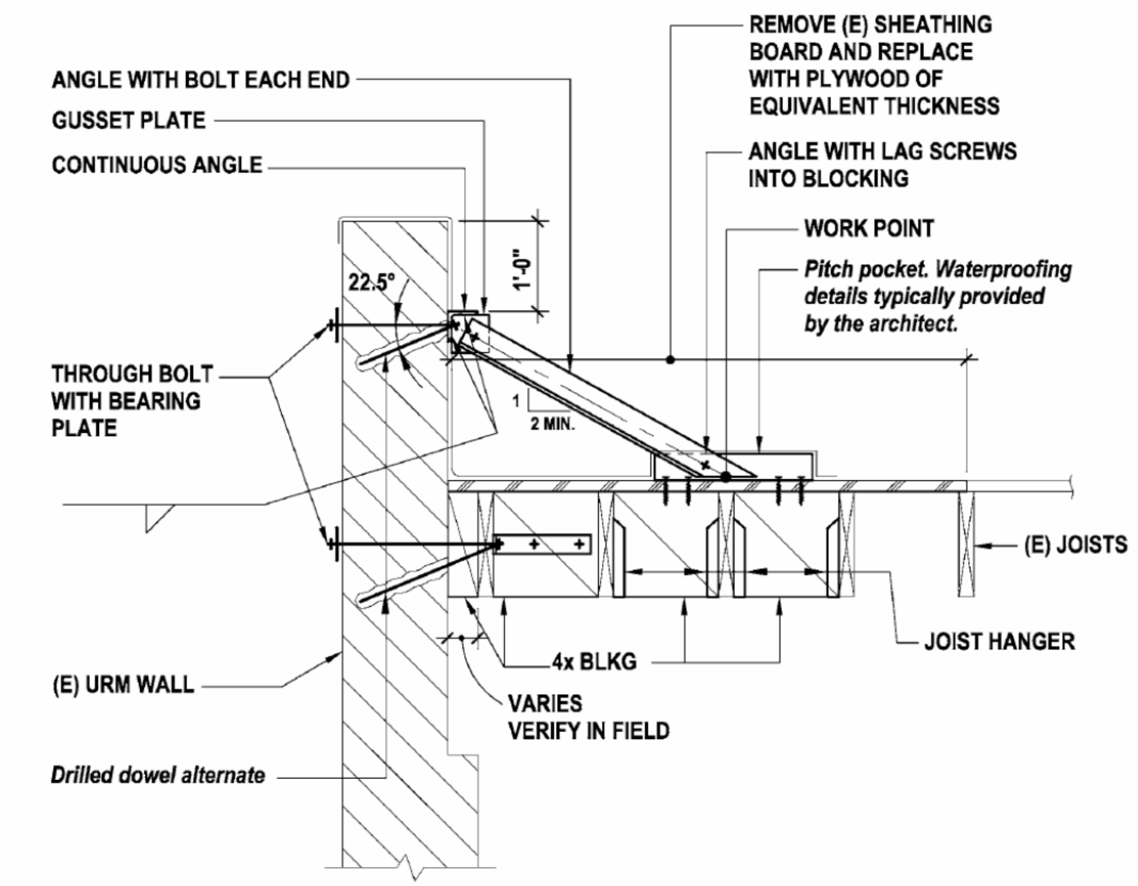



**Figure 2.2 Parapet restraint detail (FEMA 547, 2006)**

Removal of the unreinforced masonry parapet is typically combined with the addition of a concrete cap or bond beam in order to compensate for the reduction in wall axial compressive stresses and the negative effect the reduced stress level has on the capacity of roof-to-wall anchors. The concrete cap or bond beam is then typically anchored into the roof. Typical details of parapet removal and concrete cap beam are shown in Figure 2.3. See also Figure B.1(a) for an examples where this type of construction failed in the 22 February 2011 Christchurch earthquake.

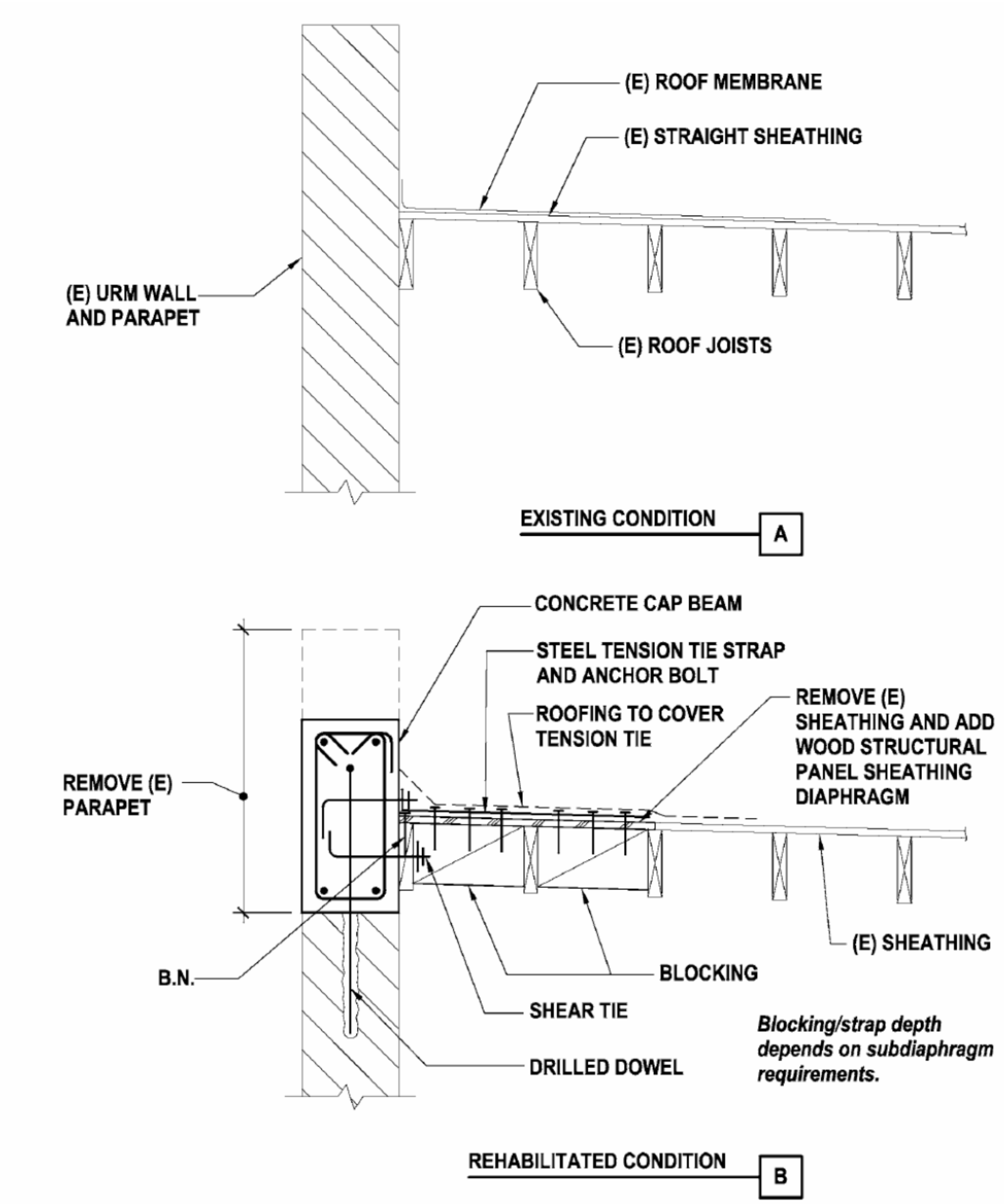
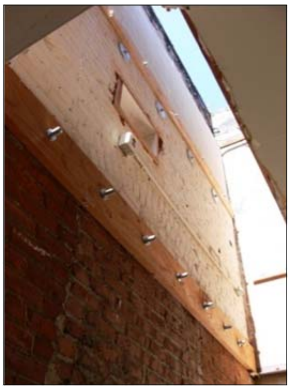



**Figure 2.3 Parapet removal and concrete cap beam installation (FEMA 547, 2006)**

### 2.3.2 Wall-to-diaphragm ties

Ties that connect masonry walls and timber floor or roof diaphragms are used to secure the separate building components together. Two examples from California that show this type of earthquake strengthening are shown in Figure 2.4.

(a) Extensive addition of wall-to-diaphragm anchorage, San Louis Obispo



(b) Extensive addition of wall-to-diaphragm anchorage, San Louis Obispo

**Figure 2.4 Addition of wall-to-diaphragm connections**

### 2.3.3 Inter-storey wall supports

The provision of inter-storey wall supports, commonly referred to as "strong-backs", is commonly used in the United States to increase the out-of-plane wall capacity of tall thin walls. Examples of this technique are shown in Figure 2.5.

(a) Installation of steel moment frames and steel strong-backs, San Louis Obispo

(b) Addition of internal strong-backs and extensive anchorage of masonry, San Louis Obispo

**Figure 2.5 Addition of strong-backs for increased out-of-plane wall capacity**

### 2.3.4 In-plane wall strengthening

Due to the commercial need for open shop fronts, moment frames are the most common way to retrofit unreinforced masonry frames without introducing obstructions to door and window openings. Numerous examples can be seen of braced frames (see Figure 2.6) and shotcrete retrofit (see Figure 2.7), as well as the application of surface bonded Fibre

Reinforced Polymer (FRP) overlays. Examples were also identified of centre-coring and the insertion of either grouted reinforcing bars or unbounded post-tensioning.

(a) Steel k-brace frames used in a multi storey URM building in San Francisco

(b) Steel moment frames used in a multi storey URM building in San Francisco

(c) Reduction of window opening to increase in-plane wall capacity

**Figure 2.6 Addition of lateral resisting systems**


(a) Internally applied shotcrete to a URM building located in San Francisco

(b) Shotcrete applied to a URM building located in San Francisco

(c) Externally applied shotcrete to a URM building located in San Francisco

**Figure 2.7 Shotcrete rehabilitation technique**

## 2.4 Summary

As shown in the previous sections and by making comparison with the information presented in section 4 of Report No. 1, it can be seen that the methods used to earthquake strengthen URM buildings in Christchurch are very similar to the techniques used in California.

# Section 3:

# Building Inspection and Data Processing Details

In this section the procedures used to collect and process information associated with damage to URM buildings following the 22 February 2011 earthquake are reported. Details are provided regarding the general building information that was collected, and parameters that were used to describe damage to individual building elements. Information is also provided regarding assessed risk levels for potential building occupants and for potential passers-by located directly adjacent to and outside the URM building. Categories used to describe various types of earthquake strengthening are then detailed, with illustrations provided for the most commonly encountered techniques. The section concludes with an explanation of how %NBS was calculated for cases where sufficient information was available to make a determination.

## 3.1 Data collection process

Commencing in March 2011 a team of researchers was deployed to document and interpret the observed earthquake damage to masonry buildings in the Canterbury region, by investigating the failure patterns and collapse mechanisms that were commonly encountered. The procedure used by Urban Search and Rescue (USAR) was adopted, where the Christchurch CBD was discretised into numbered blocks.

Building surveys were primarily external only, with all building elevations surveyed where this was safe and access was available. However, when safe and when access was available, internal building inspections were also undertaken.

Throughout the CBD numerous active demolition sites were visited and inspected. In many cases these inspections allowed the building's internal structure to be partially inspected during the demolition process, which otherwise would not have been possible due to safety considerations. This exercise also allowed for relatively straightforward collection of small building samples from building demolition sites, including: brick and mortar samples, through ties with timber assemblages, adhesive anchor rods, and cavity ties. Inspection of demolition sites also proved to be valuable when attempting to identify the seismic strengthening systems within the internal parts of the building, and when seeking to investigate the quality of earthquake strengthening installations, particularly for adhesive type anchors.

Christchurch City Council (CCC) property records were requested and reviewed in order to confidently identify cases of earthquake strengthening, with 82 URM buildings located in the Christchurch CBD selected. Time constraints restricted a greater number of records from being reviewed. However, in some cases the CCC records lacked information about earthquake assessment and strengthening, or any structural aspects of the building. Therefore only 74 sets of records were used in the study, representing 20% of the 370 URM buildings in the CBD database.

Google maps were extensively used throughout the survey for:

- Identifying building addresses, business names and building boundaries
- Providing imagery prior to the earthquake, and allowing identification of buildings and of building elevation types, for buildings that suffered extensive damage such that details were unrecognisable following the earthquake.

Post-earthquake aerial photography was extensively used throughout the damage analysis stage of this survey. In particular, post-earthquake aerial photograph was used for:

- Identification of out-of-plane cantilever type failure modes was typically made possible (see section 3.4.2)
- Identification of parapet failures and other building components in the regions of buildings otherwise not visible from the street elevation (see section 3.7).

## 3.2 Survey population

Only buildings located in the Christchurch CBD area were included within the survey reported herein, being those URM buildings located with the region confined by the four main avenues as specified in the Terms of Reference for the Royal Inquiry and reproduced herein as Appendix A. The CBD was delineated as:

- South of Bealey Avenue
- North of Moorhouse Avenue
- West of Fitzgerald Avenue
- East of Deans Avenue.

See Figure 3.1 for the location of the 370 URM building included in the database.

All surveyed buildings were constructed of load-bearing unreinforced clay brick or natural stone masonry, with no building that was constructed having a concrete frame with masonry infill being incorporated into the study. Significant effort was made to ensure that all URM buildings located in the Christchurch CBD were incorporated into the survey. However it is known that approximately 10 buildings were not incorporated into the survey database due to limited available information on these buildings. Hence it is thought that the database includes approximately 97% (370 of 380 possible) of the URM buildings located in the Christchurch CBD area.

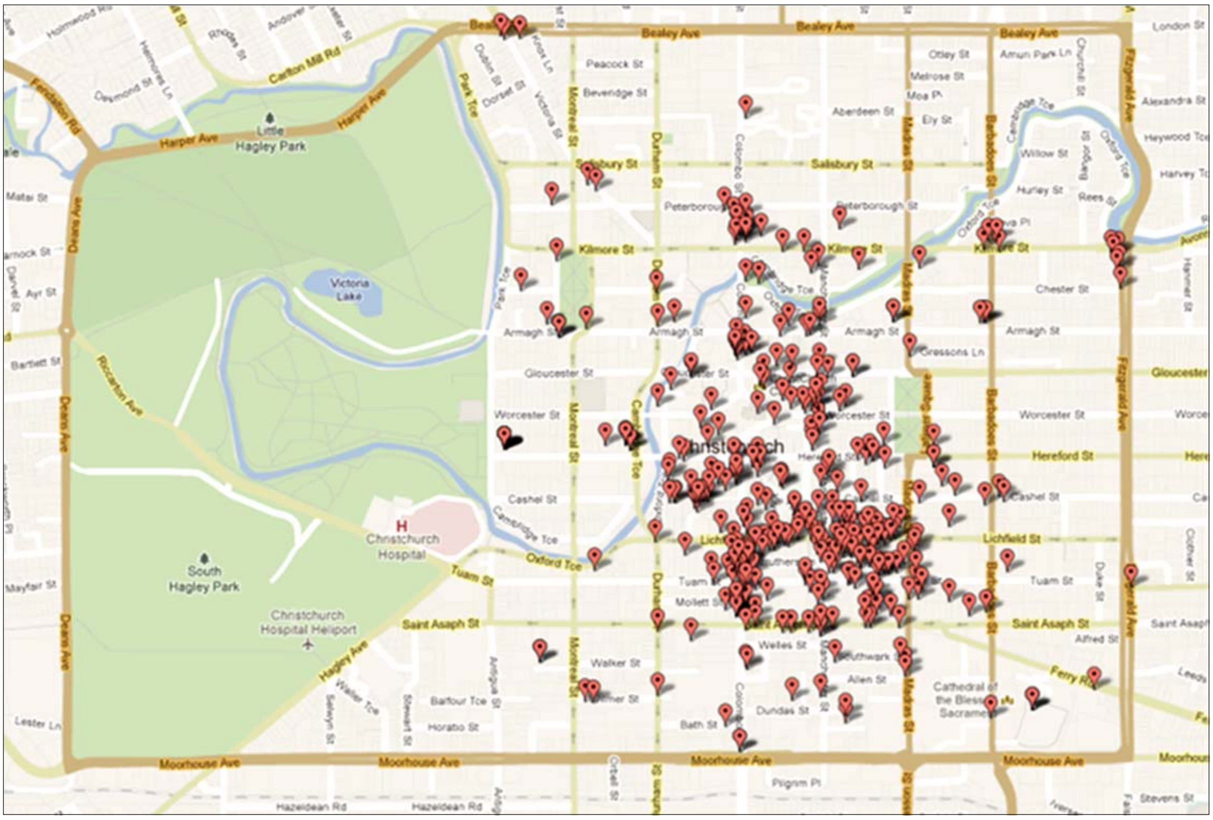



**Figure 3.1 Location of URM buildings included in this study**

## 3.3 Survey data

Survey assessment forms were specifically developed for assessment of the earthquake performance of Christchurch URM buildings. Details explaining the nature of the individual parameters that were surveyed are reviewed below.

### 3.3.1 General building information

General building information was recorded, such as:

- The address (building number and street)
- The building's original name and current name as of February 2011

- The name of the business(es) operating within the building as of February 2011
- The date of construction, if known
- Whether the building was registered with the New Zealand Historic Places Trust under the Resource Management Act 1991 or whether the building was on the Christchurch City Council list of protected buildings.

The following information was also noted:

- Number of storeys
- The presence of a basement
- The occupancy type
- The building typology (in accordance with the classification detailed in section 2.2 of Report No. 1)
- Whether the buildings was a row building or a stand-alone building
- If the building was a row building, whether it was located mid-row or end-of-row.

The level 1 placard data available from the CCC was recorded for each building and if known, the level 2 inspection placard status was also recorded. The current damage state of the building at the time of writing was noted, using the classifications of:

- Demolished
- Partially demolished
- Scheduled for demolition
- Standing.

### 3.3.2 Condition of timber diaphragms

Where possible the condition of the floor and roof diaphragms was visually assessed and a condition rating was assigned based upon the corresponding condition description as detailed in Table 3.1.

**Table 3.1 Diaphragm condition visual assessment criteria**

| Condition Rating | Condition description |
|---|---|
| Good | Timber free of Bora; little separation of floorboards; no sign of past water damage; little or no nail rust; floorboard-to-joist connection tight; coherent and unable to wobble |
| Fair | Little of no Bora; less than 3 mm of floorboard separation; little or no signs of past water damage; some nail rust but integrity still fair; floorboard-to-joist connection has some but little movement; small degree timber wear surrounding nail |
| Poor | Considerable Bora: floorboards separation greater than 3 mm; water damage evident; nail rust extensive; significant timber degradation surrounding nails; floorboard-to-joist connection appears loose and able to wobble. |

Examples of fair and poor roof diaphragms are shown in Figure 3.2.

(a) Example of fair roof diaphragm condition

(b) Example of poor roof diaphragm condition

**Figure 3.2 Diaphragm condition examples**

### 3.3.3 Building elevation details

The orientation of the main facade (or of multiple facades where present) was noted, as were details of the building elevation types. Elevation types were divided into three categories:

- Solid wall
- Solid wall with a few openings
- Perforated frame (piers and spandrels).

Where possible each building elevation was assigned a type. Examples of building elevation types are shown in Figure 3.3.

(a) Solid wall

(b) Solid wall with a few openings

(c) Perforated masonry frame, 236 Tuam Street

**Figure 3.3 Classification of building elevations**

### 3.3.4 Concrete ring beams

Buildings having concrete ring beams were identified, with an illustration of this type of construction shown in Figure 3.4. It is important to recognise that this form of construction is distinct from the 'concrete frame with masonry infill' construction type as there are no vertical concrete elements in this type of construction. Buildings constructed from concrete frame with masonry infill were excluded from this study.

**Figure 3.4 Concrete ring beams example**

### 3.3.5 Cavity construction

Cavity construction refers to a form of wall construction where an air gap is left between leaves or wythes of brick. During post-earthquake inspections cavity construction was encountered in almost half of the URM buildings surveyed in Christchurch, with the remainder having solid interconnected multi-leaf walls. However, it was identified that there were comparatively few URM buildings within the Christchurch CBD having cavity construction.

A single leaf of outer clay brick veneer is the most common type of cavity construction, with the inner section being two or more leaves thick, although double leaves on each side of the cavity were also observed. Leaves on either side of a cavity are typically held together by regularly spaced metal cavity ties, but in the case of poor connection between the leaves the outer veneer layer can 'peel' separately, as illustrated in Figure 3.5(a). It was commonly observed that some cavity ties in failed cavity walls had deteriorated and were in poor condition due to rust, as shown in Figure 3.5(b). Out-of-plane failure of the veneer was typically attributed to either the deteriorated condition of the metal ties or to pullout of the ties from the mortar bed joints due to the use during construction of weak lime mortar.

(a) Out-of-plane failure of a single leaf veneer

(b) Metal cavity ties in poor rusted condition

**Figure 3.5 Cavity wall failure**

### 3.3.6 Awnings

Awnings (also called canopies) frequently sustained significant damage due to masonry debris falling from collapsed parapets, gable ended walls, or upper storey walls. An example of a collapsed awning is shown in Figure 3.6.

**Figure 3.6 Collapse of awnings due to falling debris, at the corner of Colombo and Kilmore Streets looking north-east**

## 3.4 Wall failure mechanisms

The earthquake failure mechanisms for unreinforced masonry walls are complex, with overall behaviour customarily separated into in-plane response and out-of-plane response. Damage to wall corners also arises due to interaction between these failure modes. All three categories of wall failure mechanisms are briefly reviewed below.

### 3.4.1 In-plane damage mechanisms

In-plane wall failure modes can be associated with pier, spandrel or joint failure of masonry frames, including both diagonal tension failure and rocking and toe crushing failure for masonry piers. Horizontal bed joint sliding may also occur. These failure modes are shown schematically in Figure 3.7.

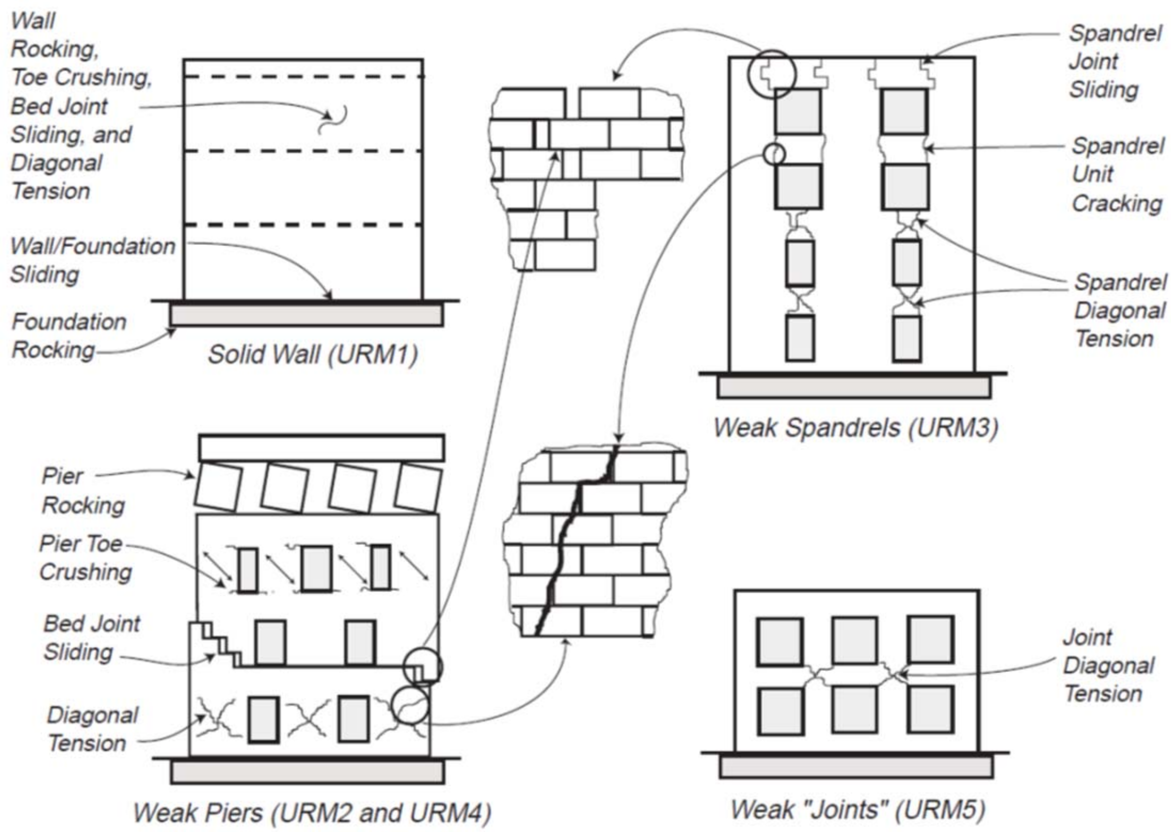
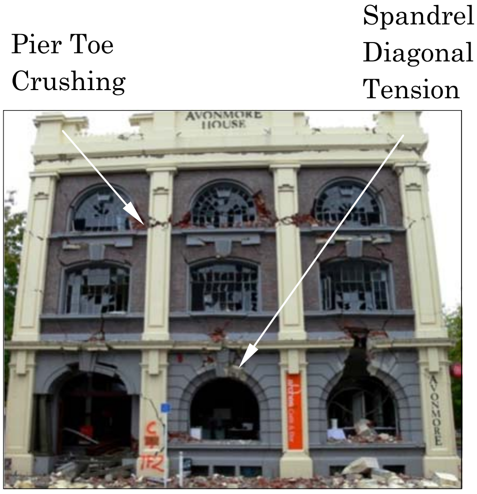
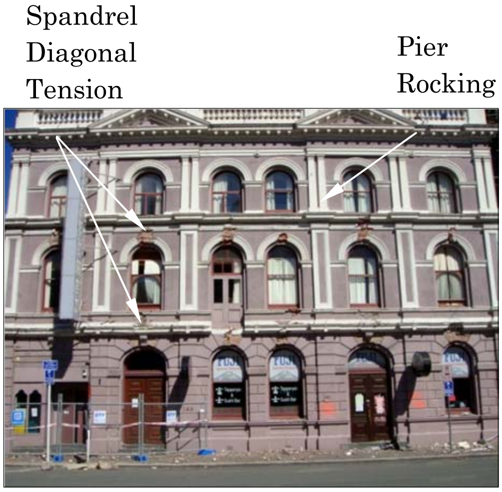



**Figure 3.7 FEMA 306 Classification of in-plane failure modes (FEMA, 1999a)**

Representative examples of in-plane failure to unreinforced masonry walls, which occurred during the 22 February 2011 earthquake, are shown in Figure 3.8.




(a) Spandrel failures, 203 Hereford Street




(b) Spandrel failures, 144 Gloucester Street

(c) Weak piers, pier rocking, 82 Lichfield Street


(d) Complete collapse of top storey due to weak piers, 84 Lichfield Street


(e) Weak piers, 156 Gloucester Street

**Figure 3.8 Examples of in-plane wall failure modes**

### 3.4.2 Out-of-plane damage mechanisms

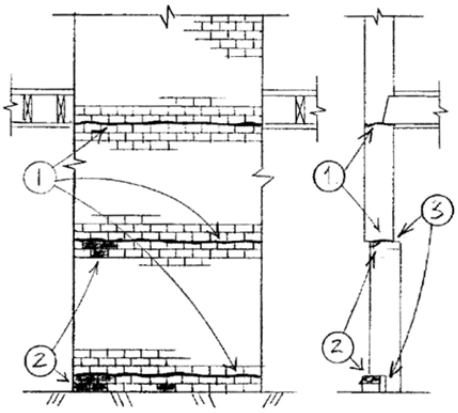
Out-of-plane wall collapse was the most commonly observed failure mode for clay brick URM buildings following the 22 February 2011 aftershock, with many two-storey buildings losing their entire front facades or upper storey walls (see Figure 3.9(a-c)).

(a) Aerial view of damage, 202-204 Madras Street

(b) North elevation damage view, 202-204 Madras Street

(c) Out-of-plane collapse of parapet and facade, 105 -109 Manchester Street


(d) One-way out-of-plane bending wall failure below a concrete ring beam (building located outside CBD)




(e) FEMA 306 classification of damage due to one-way bending failure (FEMA, 1999a)

(f) Two-way bending out-of-plane wall failure

**Figure 3.9 Out-of-plane damage types**

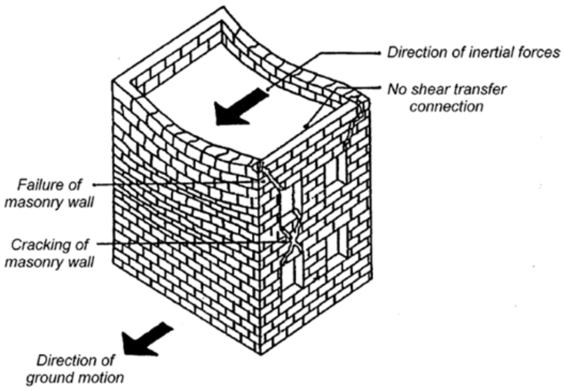
Two primary types of out-of-plane wall failures were observed:

- Vertical (or one-way) bending of the wall, which tended to occur in longer walls or walls without side supports (see Figure 3.9(d-e))
- Two-way bending, which required support of at least one vertical edge of a wall (see Figure 3.9(f)).

Cantilever type out-of-plane failure with the entire top section of a wall or building facade collapsing (see Figure 3.9(c)) was commonly observed. However, when the top section of the wall was well connected to diaphragms, failures in vertical or two-way bending were observed. Post-earthquake aerial photography was used where possible to identify the cantilever type of out-of-plane failure (see Figure 3.9(a-b)).

### 3.4.3 Wall corner damage

The intersections between perpendicular masonry walls can sustain significant damage due to the required transfer of forces at this location and the interplay between in-plane and out-of-plane deformation modes for the two walls connected at the corner. Examples of this type of damage are shown in Figure 3.10.




(a) FEMA 306 schematic of wall corner failure mechanism (FEMA, 1999a)

(b) Corner failure

**Figure 3.10 Corner failure mechanism**

## 3.5 Overall building damage level

Two scales were used to identify the overall damage observed for each building. The protocols developed by the Applied Technology Council were used because of their widespread use in past post-earthquake damage inspections, with the damage scale shown in Table 3.2.

The second damage scale adopted was that developed by Wailes and Horner (1933), which was specifically developed to describe damage to unreinforced masonry buildings. Details of this damage scale are reported in Table 3.3. Figure 3.11 shows examples of damage levels A-D using the Wailes and Horner (1933) damage scale.

**Table 3.2 ATC 38/13 General Damage Classification (ATC, 1985)**

| Classification | Replacement Value |
|---|---|
| None | 0% |
| Insignificant | 1-10% |
| Moderate | 10-30% |
| Heavy | 30-60% |
| Major | 60-100% |
| Destroyed | 100% |

**Table 3.3 Wailes and Horner (1933) Damage Scale**

| Damage level | Damage description |
|---|---|
| A | Undamaged or Minor Cracking, No Significant Structural Damage, Minor Veneer Damage |
| B | Parapet Failure or Separation of Veneer, Major Wall Cracking and Interior Damage |
| C | Failure of Portion of Exterior Walls, Major Damage to less than 50% of walls |
| D | Major Damage to more than 50% of walls |
| E | Unrepairable Damage, Demolition Probably Appropriate |

(a) Damage classification - Insignificant
Damage level – A
(292 Kilmore Street)

(b) Damage classification - Moderate
Damage level – B
(200 Madras Street)

(c) Damage classification - Heavy
Damage level – C
(120 Manchester Street)

(d) Damage classification - Destroyed
Damage level – D
(202 Hereford Street)

**Figure 3.11 Damage classification examples**

## 3.6 Overall elevation damage level

The level of damage to each building elevation was based upon an adaption of the ATC 38/13 classification detailed in Table 3.2, using the following damage levels:

- unknown
- non-visible
- insignificant
- moderate
- heavy
- extreme

Also, the orientation of each building elevation was noted in order to identify the influence of directionality on wall failure modes.

## 3.7 Damage to parapets

Parapets are parts of URM construction that project above the roof of the building. When subjected to lateral loads, if unrestrained the parapet acts as a vertical cantilever which potentially rocks on the base support at the roof line. The level of damage to parapets and the parapet orientation were recorded. For each orientation the damage was classified as:

- none
- minor
- moderate (see Figure 3.12(a))
- heavy (see Figure 3.12(b))
- partial collapse
- full collapse.

(a) Moderate damage to parapet (b) Heavy damage to parapet

**Figure 3.12 Damage level to parapets due to rocking**

## 3.8 Other recorded damage

The level of damage due to falling debris from adjacent building was recorded and classified as:

- none
- minor
- moderate
- severe.

Damage due to pounding from adjacent building (see Figure 3.13) was also assessed and recorded.


**Figure 3.13 Example of building pounding causing in-plane failure, High Street**

The level of liquefaction, lateral spreading and permanent ground settlements at each site was also recorded for each building. However, the influence of ground effects upon observed damage (as distinct from earthquake shaking effects) is not specifically addressed in this report.

## 3.9 Temporary shoring and securing

For URM buildings damaged in the September 2010 earthquake, temporary shoring was commonly used to prevent further out-of-plane wall damage or collapse. Hence the performance of temporary shoring was assessed following the major aftershocks. Figure 3.14 shows post-September 2010 shoring that assisted in preventing collapse of URM buildings in February 2011. A critical review of the performance of temporary securing and shoring is outside the scope of this report.

(a) Extensive steel temporary shoring of a URM building following 4 Sept. 2010 Darfield earthquake, (building located outside CBD)


(b) Temporary securing work on a URM building following 4 Sept. 2010 Darfield earthquake, corner of Gloucester and Manchester Street

**Figure 3.14 Examples of temporary shoring and securing**

## 3.10 Risk levels to building occupants and passers-by

The risk level or hazard for building occupants was estimated depending on the extent of observed damage, with the risk assigned to each building using the classifications of:

- <u>unlikely</u> risk of death or serious injury
- <u>likely</u> risk of death or serious injury
- <u>near certain</u> risk of death or serious injury.

The same exercise was undertaken for the occupants (frequently referred to herein as 'passers-by) of the public spaces that were located directly outside and adjacent to the building. Illustrations of this hazard to passers-by are shown in Figure 3.15.

## 3.11 Categories of earthquake strengthening

The various forms of earthquake strengthening of unreinforced masonry buildings that were encountered during the survey were divided into three general categories for ease of assessment and the convenient interpretation of data. These three categories were: Parapet restraints, Type A earthquake improvements, and Type B earthquake improvement. Each category of earthquake strengthening is further discussed below.

### 3.11.1 Parapet restraints

Recognising the obvious hazard that parapets pose to building occupants and passers-by (see section 3.10), it follows that the installation of parapet restraints is one of the earthquake strengthening techniques that is most commonly encountered in unreinforced masonry buildings.

(a) Entire collapse of a building on to a road, near certain risk to building occupants and to public space, 225-227 Manchester Street

(b) Partially collapsed building poses a near certain risk to building occupants and to public space , 194 Gloucester Street

(c) Partially collapsed building poses a unlikely risk to building occupants and near certain risk to public space, 195 Armagh Street


(d) Partially collapsed building poses a likely risk to building occupants and near certain risk to public space, 202 Hereford Street

**Figure 3.15 Examples of risk levels to building occupants and passers-by**

Typically observed methods used to provide parapet restraint include but are not limited to:

- lowering of the parapets
- the addition of a concrete ring beam
- bracing of the parapet back into the roof structure via steel members connected to the parapets and secured into the masonry using either adhesive anchors or through ties.

### 3.11.2 Type A earthquake improvements

Earthquake improvement techniques that assisted in connecting the walls and diaphragms of unreinforced masonry buildings were categorised as Type A earthquake improvements. Type A earthquake improvements were:

- Securing/strengthening URM building elements such as gable ends (excluding parapet restraints, which were considered separately as detailed in section 3.11.1).
- Installing connections between the walls and the roof and floor systems of the URM building so that walls no longer respond as vertical cantilevers secured only at their base. An example of this connection type is shown in Figure 3.16(a)
- Stiffening of the roof and/or floor diaphragms.

Further details of these Type A earthquake strengthening techniques are provided below.

#### 3.11.2.1 Gable end wall restraints

A gable is the triangular portion of wall that is typically located at the ends of URM buildings. As with parapets, because it is widely known that gable end walls are prone to failure, it is not uncommon to find some form of restraint provided to inhibit out-of-plane failure during an earthquake. Where present, the type of gable end wall restraint was identified and recorded, using three categories:

- through ties
- adhesive anchors
- original (referring to restraints installed at the time of construction)

Examples of gable end wall restraints are shown in Figure 3.16.

#### 3.11.2.2 Wall to diaphragms connection improvement

The addition of positive connection between masonry walls and the floor or roof diaphragms assists the URM building to respond as a single complete structure (rather that a set of poorly connected structural elements) when subjected to earthquake loading. This type of earthquake strengthening is typically done by the installation of:

- Through tie anchors (see Figure 3.17(a))
- Adhesive anchors (see Figure 3.17(b)).

#### 3.11.2.3 Floor and roof diaphragm improvement

Similar to the installation of wall to diaphragm connections, the strengthening and stiffening of flexible timber diaphragms improves the earthquake response of URM buildings. Diaphragm improvement techniques, where identified, were noted in the survey and typically consisted of:

- Addition of steel bracing (see Figure 3.18(a))
- Addition of horizontally oriented steel brace frames (see Figure 3.18(b))
- Plywood diaphragm overlays (see Figure 3.18(c))
- Addition of a concrete floor slab.

Any diaphragms not readily described using the above classifications were referred to as ‘other’.

(a) Restraint used at the time of constriction

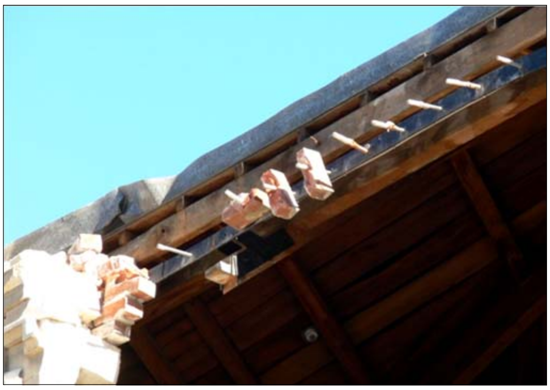
(b) Restraint used at the time of constuction

(c) Example of through tie anchorage

(d) Gable collapse due to inadequate connections

**Figure 3.16 Examples of gable restraints**

(a) Example of through ties

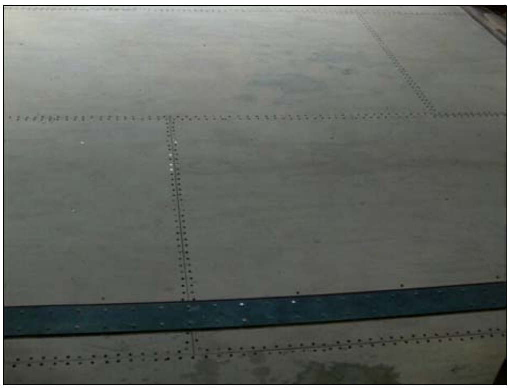


(b) Example of adhesive diaphragm anchors

**Figure 3.17 Diaphragm to wall connections**

(a) Roof diaphragm improvement using steel brace frames

(b) Roof diaphragm improvement using steel braces



(c) Floor diaphragm improvement using plywood overlays

(d) Floor diaphragm improvement using overlays and steel framing

**Figure 3.18 Example of diaphragm improvement techniques**

### 3.11.3 Type B earthquake improvements

Type B earthquake improvements were defined as strengthening techniques that sought to strengthen masonry walls and/or introduce added structure to supplement or replace the earthquake strength provided by the original unreinforced masonry structure.

Examples of Type B earthquake improvement are:

- Strong-backs installed either internally or externally
- Steel moment frames
- Steel brace frames
- Concrete moment frames
- Addition of cross walls
- Shotcrete
- FRP
- Post tensioning
- Other

In this study the term shotcrete was used to not only describe added concrete walls that have been 'shot' onto the URM wall using high pressure pumping equipment, but also cast concrete walls. This decision was made in order to avoid having an increased number of classifications that each had a minimal number of recorded cases of implementation.

Further background details regarding these Type B strengthening techniques are provided in section 4 of Report No. 1.

### 3.11.4 **Retrofit level (%NBS)**

The level of seismic improvement (i.e. %NBS) was obtained either from CCC records, from personal communication with building owners, engineers and heritage personnel, or in some cases where sufficient seismic details about the building were known, was based upon estimation. Assessed CCC records showed that most engineering reports provided the level of strengthening in terms of design PGA values. To convert the level of earthquake strengthening for a building to the current (2011) %NBS values the procedure outlined below was used.

From NZS1170.5:2004, the horizontal design action coefficient is:

$$C_d(T_1) = \frac{C(T_1)S_p}{k_\mu}$$

where

$$C(T) = C_h(T)\,Z\,R\,N(T,D)$$

Assuming T = 0.5 s, and adopting soil class type D (which is typical for URM buildings located in the Christchurch CBD), $C_h(0.5) = 3.00$, $Z = 0.22$ (now revised to 0.3), $R = 1.0$, and $N(T,D) = 1.0$.

Therefore $C(0.5) = 3.00 \times 0.22 \times 1.0 \times 1.0 = 0.66$.

Then $k_\mu = \frac{(\mu-1)T_1}{0.7} + 1$ for $T_1 < 0.7$s and soil class D and $S_p = 1.3 - 0.3\mu$

Using the recommendations from NZSEE (2011) $k_\mu = 1.2$, and therefore:

$\mu = (1.2 - 1) \times \frac{0.7}{0.5} + 1 = 1.28$ and $S_p = 1.28 - 0.3 \times 1.3 = 0.89$ so $C_d(0.5) = \frac{0.66 \times 0.89}{1.28} = 0.46$

Therefore, to calculate the seismic strengthening level (%NBS) of a building given the design PGA values:

$$\frac{\text{Retrofit design PGA}}{C_d(0.5)} \times 100\% = \%\text{NBS}$$

# Section 4:

# General Damage and Demolition statistics

In this section the general analysis and interpretation of the available survey data for 370 URM buildings located in the Christchurch CBD area is presented. The presented information includes a critique of the general characteristics of the surveyed buildings, followed by a review of damage statistics for 368 buildings (damage data was not recorded for two URM buildings). Currently available statistics regarding the number of URM buildings formerly located in the Christchurch CBD that have been or are scheduled to be demolished is also provided.

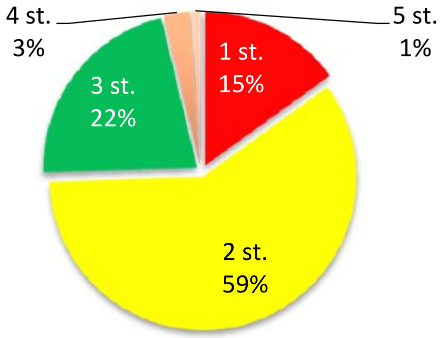
It is known that approximately 10 buildings located between Park Terrace and Deans Ave (in the Hagley Park area) were not incorporated into the database due to limited available information on these buildings. Hence the database is thought to contain approximately 97% (370 of 380 possible) of all URM buildings located in the Christchurch CBD. It is emphasised that data for additional URM buildings surrounding the Christchurch CBD, totalling approximately 250 additional URM buildings, has also been collected but has been excluded from the analysis reported here. This decision was made in consultation with the Royal Commission in order to expedite the release of this report.

## 4.1 General building characteristics

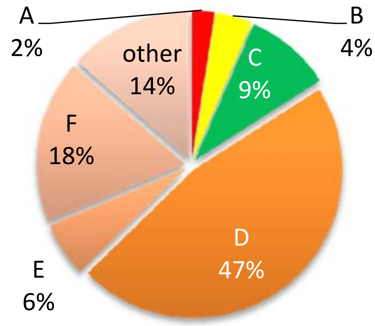
In this section the general characteristics of the 370 surveyed buildings are reported. Where appropriate, comparisons are made between the URM building stock in the Christchurch CBD and the general characteristics of the national URM building stock as detailed in Report No. 1.

### 4.1.1 Number of storeys

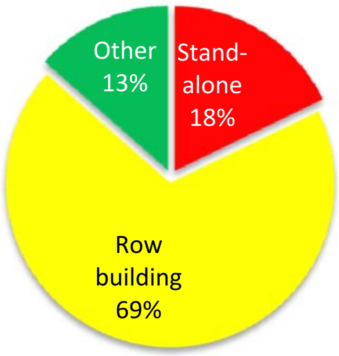
The distribution of above ground number of storeys is presented in Figure 4.1(a), where it can be seen that 59% of the surveyed buildings had two storeys and 22% of buildings surveyed had 3 storeys. Consequently two storey buildings were most prevalent and 85% of all URM buildings surveyed were multi-storey. This distribution of storey height differs from the national average as detailed in Report No. 1, where it was reported that approximately 70% of all URM buildings are thought to be single storey, but is readily explained by recognising that the surveyed buildings were exclusively from the CBD area, where building density is high.



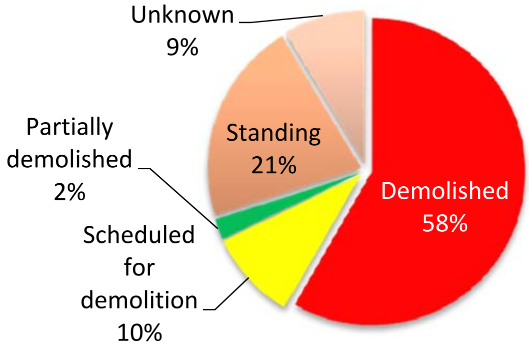

(a) Number of storeys



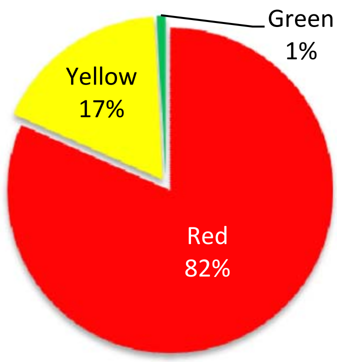

(b) Building typologies




(c) Row buildings




(d) State of buildings




(e) Level 1 placarding data (known building placards only, 337 buildings total)

**Figure 4.1 General building characteristics**

### 4.1.2 Building typologies

The distribution of all the surveyed buildings into their respective typologies (see section 2.2 of Report No. 1 for a review of typologies) is illustrated in Figure 4.1(b). Typology D (two storey row buildings) was the predominant typology in the surveyed CBD building stock. Figure 4.1(c) shows the distribution of row buildings and stand-alone buildings, again illustrating the predominance of URM row buildings in the CBD. Of the 254 (69%) row buildings, the distribution between buildings classified as being either end-of-row or mid-row was approximately equal, with 130 (51%) being end-of-row buildings and 124 (49%) being mid-row buildings.

### 4.1.3 Building elevations

A total of 922 building elevations were surveyed, where 'building elevation' refers to an exterior building face. Hence two building elevations exist for a mid-row building (front and back), three building elevations exist for an end-of-row building (front, back, side) and four building elevations exist for a stand-alone building (front, back, two sides). When accounting for the mix of row and stand-alone buildings as discussed in section 4.1.2, the 922 surveyed building elevations is thought to represent approximately 84% of all building elevations associated with the 370 surveyed URM buildings. Missing data was due to restricted access for some buildings. The distribution of elevation types is presented in Table 4.1, showing that 391 (42%) of the surveyed elevations were perforated masonry frames and a further 383 (42%) building elevations were walls that were either solid (without openings) or had few openings.

**Table 4.1 Data distribution by elevation type**

| Elevation type | No. of elevations | % of elevations |
|---|---|---|
| Perforated frame (piers/spandrels) | 391 | 42% |
| Solid wall | 120 | 13% |
| Solid wall with a few openings | 263 | 29% |
| Not identified | 148 | 16% |
| **Total** | **922** | **100%** |

As shown in Table 4.2, 216 (55%) of the perforated frames had piers with an aspect ratio (ratio of pier vertical height divided by pier horizontal length) greater than 1 (classified as tall piers), with only 36 (9%) of the perforated frames having piers with an aspect ratio of less than 1 (classified as squat piers).

**Table 4.2 Pier aspect ratio type**

| Pier type | No. of elevations | % of elevations |
|---|---|---|
| Tall | 216 | 55% |
| Squat | 36 | 9% |
| Mix | 53 | 14% |
| Unspecified | 86 | 22% |
| **Total** | **391** | **100%** |

### 4.1.4 Concrete ring beams and cavity construction

A relatively small number of buildings having concrete ring beam construction (see section 3.3.4) were identified, and likewise few buildings having cavity type construction (see section 3.3.5) were identified (note that this latter comment is inconsistent with observations for URM buildings located outside the CBD but not reported herein, where a significant proportion of all buildings were identified to have cavity construction). Of the 370 buildings surveyed, in most cases it was not possible to identify whether cavity construction had been employed due to the extensive use of external render shielding identification of the brick bond pattern, but of the 134 buildings where a definitive assessment was possible it was established that 94 (70%) buildings were of solid wall construction and 40 (30%) buildings had cavity wall construction (see Table 4.3).

**Table 4.3 Prevalence of cavity wall construction in the Christchurch CBD**

| Cavity construction | No. of buildings | % of buildings | % where wall type known |
|---|---|---|---|
| Yes | 40 | 11% | 30% |
| No | 94 | 25% | 70% |
| Unknown | 236 | 64% | - |
| **Total** | **370** | **100%** | **100%** |

### 4.1.5 Construction material type

333 (90%) of the surveyed URM buildings were constructed using clay brick masonry as the principal construction material type, with the remaining 37 (10%) being constructed of either natural stone or a combination of clay brick and natural stone masonry. The distribution of the construction material of all surveyed buildings is shown in Table 4.4.

**Table 4.4 Construction material type**

| Material type | No. of buildings | % of buildings |
|---|---|---|
| Clay brick | 333 | 90% |
| Stone | 13 | 4% |
| Clay brick and stone | 24 | 6% |
| **Total** | **370** | **100%** |

### 4.1.6 Construction date

The year of construction of each building was recorded where possible, and ranged from 1864 to 1930. This information was mostly available for buildings that are registered with the New Zealand Historic Places Trust (NZHPT) and/or on the Christchurch City Council (CCC) list of protected buildings, or cases where the construction year was displayed on the building.

The year or decade of construction was recorded for 154 (42%) buildings. The distribution of construction date by decade is shown in Table 4.5, showing that for the data available the construction of URM buildings peaked in the Christchurch CBD during the first decade of the 20th century. The data in Table 4.5 are consistent with the documented early history of Christchurch as reported in section 1.2 of Report No. 1.

**Table 4.5 Construction date by decade**

| Decade of construction | No. of buildings | % of total buildings | % of building for which date known |
|---|---|---|---|
| 1860-1869 | 6 | 2% | 4% |
| 1870-1879 | 13 | 4% | 8% |
| 1880-1889 | 24 | 7% | 15% |
| 1890-1899 | 20 | 5% | 13% |
| 1900-1909 | 55 | 15% | 36% |
| 1910-1919 | 24 | 6% | 16% |
| 1920-1929 | 12 | 3% | 8% |
| unknown | 216 | 58% | - |
| **Total** | **370** | **100%** | **100%** |

### 4.1.7 Occupancy type

As shown in Table 4.6 the most common occupancy type was commercial/offices, with 323 (87%) buildings being assigned this classification. This finding is consistent with the fact that the survey was confined to the CBD area, but is also likely to be true for the national URM building stock.

**Table 4.6 Occupancy type**

| Occupancy type | No. of buildings | % of buildings |
|---|---|---|
| Commercial/offices | 323 | 87% |
| Dwelling | 4 | 1% |
| Mixed | 13 | 4% |
| Industrial | 11 | 3% |
| School | 2 | 1% |
| Religious | 8 | 2% |
| Community | 9 | 2% |
| **Total** | **370** | **100%** |

## 4.2 Damage assessments

In this section two methods for describing the extent of damage to the entire URM building population incorporated within the study are presented. Damage assessment statistics are presented for 368 buildings as damage data was not recorded for two URM buildings.

### 4.2.1 Placard data from level 1 inspections

Available placard data specifically associated with the 22 February 2011 earthquake was obtained from Christchurch City Council, and was supplemented where appropriate by observed placards identified during surveying. Placard data was obtained for a total of 337 (91%) buildings, as shown in Table 4.7. In Figure 4.1(e) the distribution of building placard levels is plotted. The unknown category exists because there were some gaps in the data obtained from CCC. It is evident that the majority of URM buildings in the Christchurch CBD were given a red placard following the 22 February 2011 earthquake, which differs markedly from the placard distribution (for 595 URM buildings located both within and outside the CBD) for the 4 September 2010 earthquake as reported in Figure 3.5 of Report No. 1, where 47% were assigned Green, 32% were assigned Yellow and 21% were assigned Red.

**Table 4.7 URM building placard data for 22 February 2011 earthquake**

| Level 1 Placard | No. of buildings | % of buildings |
|---|---|---|
| Green | 3 | 1% |
| Yellow | 59 | 16% |
| Red | 275 | 74% |
| Unknown | 33 | 9% |
| **Total** | **370** | **100%** |

### 4.2.2 General damage classification scheme

The overall damage to each surveyed building was visually assessed using two damage scales, as further detailed below. The ATC scale has previously been used widely for a variety of structural forms whereas the scale by Wailes and Horner has previously been used specifically for URM buildings. Reviews of various damage classification schemes relevant to this study have been published by EERI (2003b) and Spence (2010).

Martel (1936) published details on a survey of 1,261 URM buildings following the 10 March 1933 Long Beach earthquake and more recently Rutherford & Chekene (1990) and Lizundia et al. (1993) have presented results of a survey of 2,007 unreinforced masonry buildings in San Francisco in the months after the 1989 Loma Prieta earthquake. For the recording of damage states following the 1989 Loma Prieta earthquake the ATC-13 (1985) and Wailes and Horner scales were adopted, and for consistency the same procedure was used in the current study. In order to expedite release of the current report no effort has yet been made to correlate the Christchurch damage data sets with other post-earthquake damage data sets previously collected elsewhere, but it is the intent that in time this exercise will be completed. Notably, one conclusion from prior studies was that the quality of recorded data was negatively influenced by the large number of people responsible for data collection whereas in the current study the same personnel were responsible for collecting the entire data set.

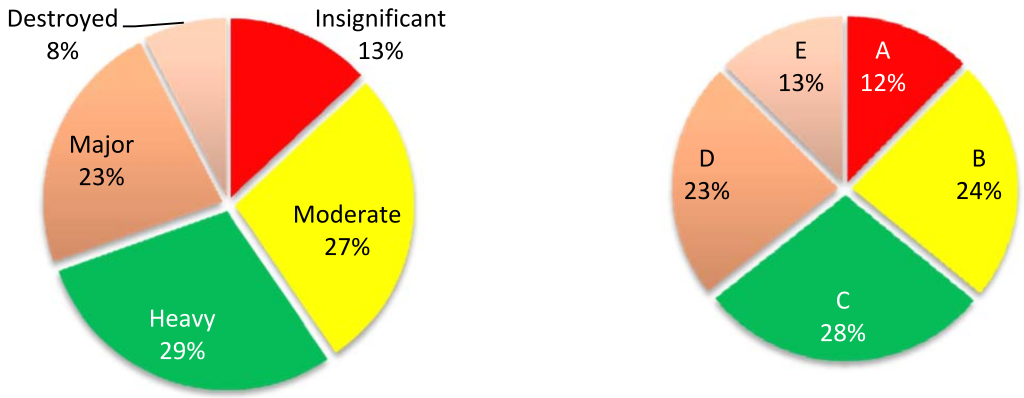
#### 4.2.2.1 ATC 38/13 General Damage Classification Scheme

The Applied Technology Council (ATC) has a general damage classification scheme[6] as detailed in Table 4.8, based upon an assessment of the overall scale of damage (ATC, 1985). This scheme has been adopted because of the widespread usage of ATC protocols in post-earthquake building assessments.

**Table 4.8 ATC 38/13 general damage classification**

| Classification | Associated damage value | No. (%) of buildings |
|---|---|---|
| None | 0% | 0 |
| Insignificant | 1-10% | 48 (13%) |
| Moderate | 10-30% | 101 (27%) |
| Heavy | 30-60% | 107 (29%) |
| Major | 60-100% | 84 (23%) |
| Destroyed | 100% | 28 (8%) |
| **Total** | | **368 (100%)** |

As can be seen in Table 4.8 and Figure 4.2(a), the ATC scale reported damage in excess of moderate for 219 (60%) of all surveyed buildings, and damage in excess of insignificant for 320 (87%) of all surveyed buildings.




(a) ATC 38/13 Classification (b) Wailes and Horner Scale

**Figure 4.2 Damage level using two different classifications schemes**

[6] See http://www.atcouncil.org/pdfs/atc38assmtfrm.pdf

#### 4.2.2.2 Wailes and Horner General Damage Classification Scheme

As discussed earlier, the Wailes and Horner (1933) damage scale was specifically developed for the post-earthquake assessment of URM buildings, and is based upon descriptions of URM building damage instead of an overall assessment. Damage statistics as classified using this scale are reported in Table 4.9 and in Figure 4.2(b). It can be seen that 322 (88%) of all buildings were classified as having suffered significant structural damage (Level B, C, D or E) when using the Wailes and Horner damage scale.

**Table 4.9 Wailes and Horner damage scale**

| Damage Level | Damage description | No. (% ) of buildings |
|---|---|---|
| A | Undamaged or Minor Cracking, No Significant Structural Damage, Minor Veneer Damage | 46 (12%) |
| B | Parapet Failure or Separation of Veneer, Major Wall Cracking and Interior Damage | 88 (24%) |
| C | Failure of Portion of Exterior Walls, Major Damage to less than 50% of walls | 106 (28%) |
| D | Major Damage to more than 50% of walls | 86 (23%) |
| E | Unrepairable Damage, Demolition Probably Appropriate | 42 (13%) |
| **Total** | | **368 (100%)** |

#### 4.2.2.3 Correlation between the two damage scales

As noted above, it was determined that when using the ATC 38/13 damage classification there were 320 (87%) surveyed buildings that suffered damage in excess of insignificant, and when using the Wailes and Horner damage scale there were 322 (88%) surveyed buildings that suffered significant structural damage (Level B, C, D or E). Clearly this comparison indicates a high level of correlation between the two survey methods. This high correlation can be further identified by considering the two charts in Figure 4.2, where it can be seen that comparable numbers of buildings were assigned for each of the incremental damage levels within the two scales.

## 4.3 Comparing assessed damage level and building placard data

A plot of overall building damage against available placard data (from level 1 inspections as reported in section 4.2.1) is shown in Figure 4.3. The prevalence of the red placard assignments to URM buildings having insignificant or moderate damage shows that assigned placard levels tended to be a conservative estimate of structural damage. Note that both the building damage assessments and the level 1 inspections for placards were primarily based on external inspections only.

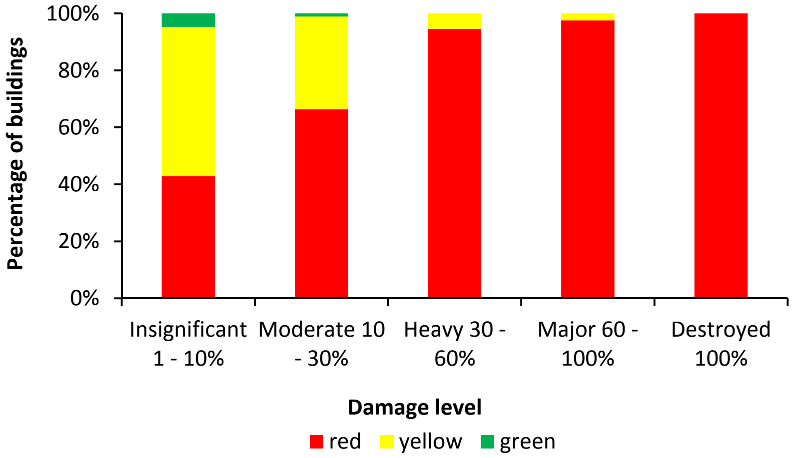



**Figure 4.3 Plot of damage levels vs. placard data (level 1)**

## 4.4 Damage for different building forms

The ATC 38/13 damage classification was used to investigate any possible relationship between the number of stories and the level of damage (see also Figure 4.1(a) for the distribution of storey heights for the entire URM building population). As shown in Figure 4.4, and specifically neglecting the case for 5 storey buildings due to this category representing only 1% of the building population, it was concluded that there was no distinct relationship between overall building height and the level of sustained building damage. Notably the same conclusion was reached following the 4 September 2010 Darfield earthquake (see Figure 3.5(c) of Report No. 1).

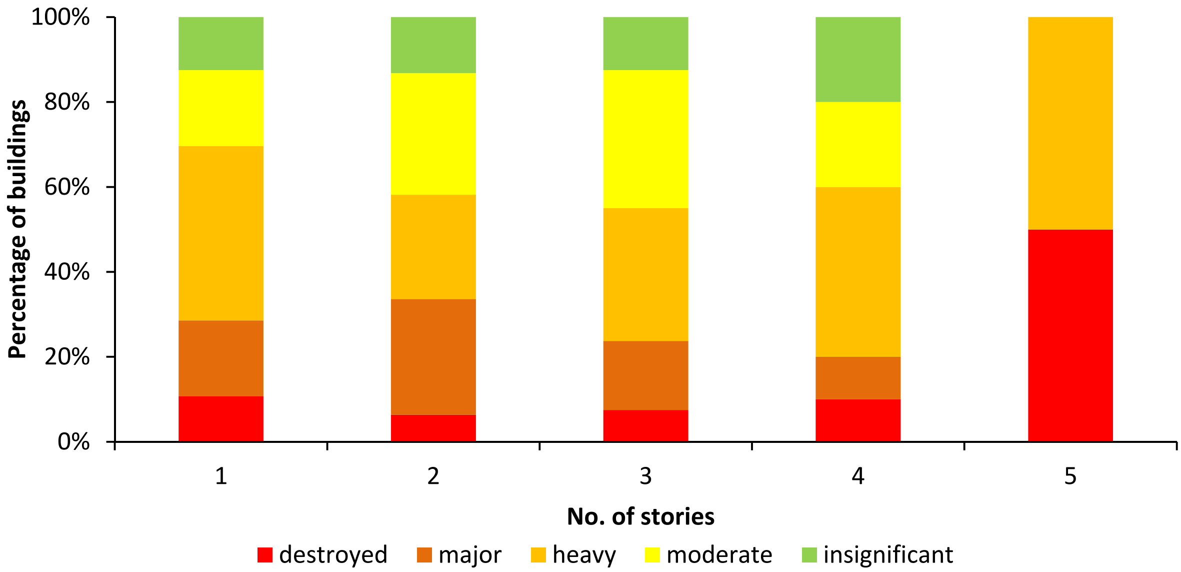
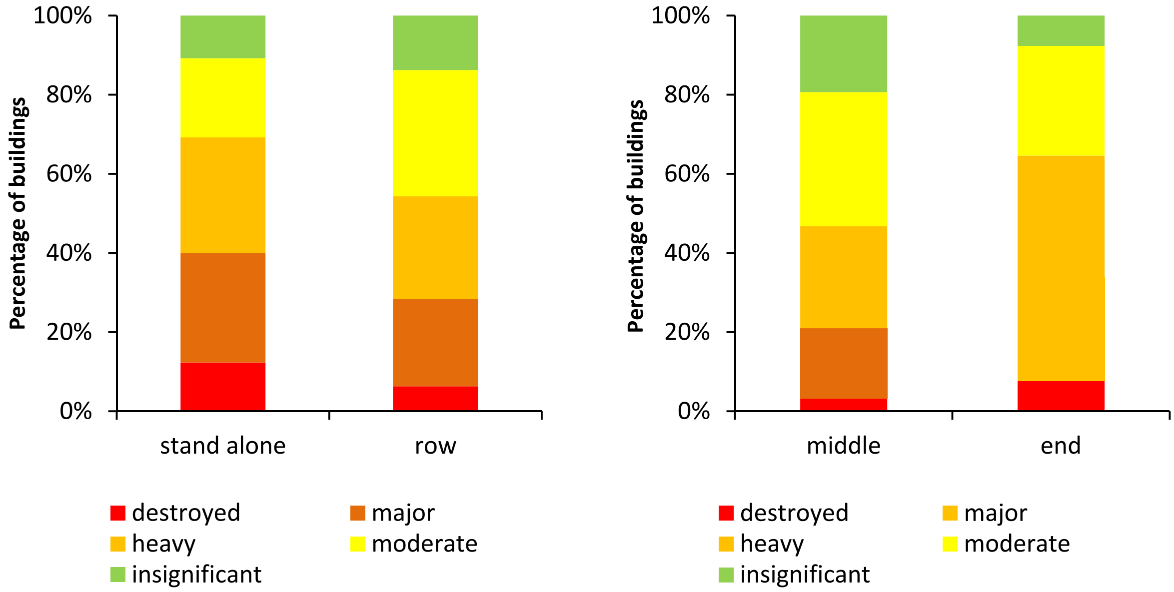



**Figure 4.4 ATC 38/13 damage classification for number of stories**

Next the level of damage was investigated by separating the damage data into stand-alone versus row buildings (see Figure 4.5(a)), where it was established that there was a greater degree of damage to stand-alone buildings than to row buildings. Similarly, the performance of row buildings was considered based upon damage correlated to whether the building was located mid-row or end-of-row (see Figure 4.5(b)), where it was found that greater damage was sustained to end-of-row buildings. These findings are consistent with the generally held view that mid-row buildings are somewhat protected from damage by the end-of-row buildings (occasionally referred to as 'bookend' behaviour) and similarly that the practice of constructing multiple URM buildings connected together provides additional robustness when compared to the earthquake performance of stand-alone URM buildings.




(a) Damage levels for stand-alone vs. row buildings

(b) Damage levels for row buildings depending on position

**Figure 4.5 ATC 38/13 damage classification for stand-alone and row buildings**

Finally, the two analyses discussed above were amalgamated to identify the associated damage levels for different building typologies (see Figure 4.6). Recalling that Typology A and C are 1 and 2 storey stand-alone buildings and that Typology B and D are 1 and 2 storey row buildings (see section 2.2 of Report No. 1 for a full description), the damage trends discussed above lead to the conclusion that Typology B and D buildings should exhibit less damage than for other typologies. This trend is shown in Figure 4.6.

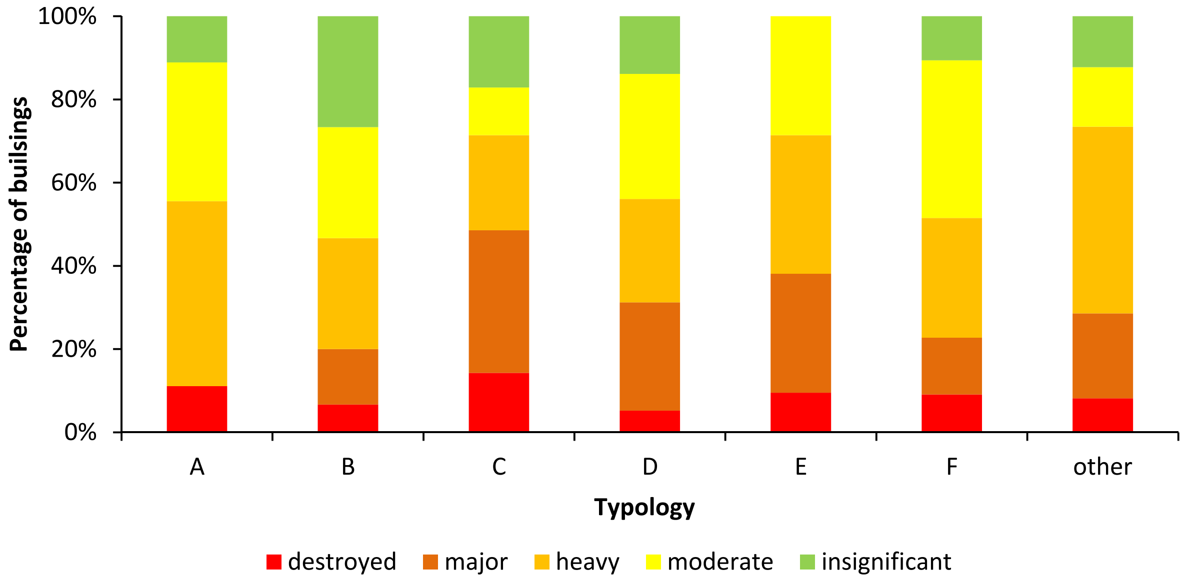



**Figure 4.6 ATC 38/13 damage classification for building typology**

## 4.5 Damage to individual building elevations

The damage levels for individual building elevations using the ATC damage scale reported in section 4.2.2.1 are presented in Table 4.10. Also reproduced in the far right column of Table 4.10 are the damage statistics previously reported in Table 4.8 for entire buildings.

**Table 4.10 Damage levels for all elevations**

| Elevation damage level | No. (%) of elevations | No. (%) of buildings |
|---|---|---|
| None visible | 68 (7%) | 0 |
| Insignificant | 127 (14%) | 48 (13%) |
| Moderate | 237 (26%) | 101 (27%) |
| Heavy | 204 (22%) | 107 (29%) |
| Extreme | 286 (31%) | 112 (31%) |
| **Total** | **922 (100%)** | **368 (100%)** |

It is evident in Table 4.10 that there is a strong correlation between the percentages of damage recorded for each damage level when comparing the data for individual building elevations and overall building response, as would be expected. By careful scrutiny of the photographs of each building elevation it is expected that it will eventually be possible to report the distribution of different failure modes that led to the damage levels reported in Table 4.10, but this analysis has yet to be undertaken.

## 4.6 Damage to awnings

Of the 370 URM buildings surveyed in the CBD only 173 (47%) buildings were identified as having awnings. Of these, 68 (39%) buildings had failed awnings due to falling debris from parapets and walls above.

## 4.7 Wall corner damage

Of the 370 buildings surveyed in the CBD, 108 (29%) buildings were identified as having wall corner damage failure (see section 3.4.3 for description) and 262 (71%) were identified as having no wall corner damage.

## 4.8 Damage due to pounding

Approximately 12% of URM buildings were identified as having some degree of damage due to pounding with neighbouring building(s). Comprehensive data on the location and extent of pounding damage in URM buildings was collected, but currently is not sufficiently collated for inclusion herein.

## 4.9 The performance of temporary securing and shoring

Following the original 4 September 2010 Darfield earthquake a number of buildings had temporary shoring[7] installed in preparation for or during repair works. Of the 370 buildings surveyed, 49 (13%) buildings had visible temporary shoring. Of these 49 shoring installations, 22 (45%) installations used steel braces, 25 (51%) installations used timber braces and 2 (4%) installations used other shoring methods. The performance of the shoring was assessed, and it was determined that installed shoring prevented collapse in 37 (75%) cases.

In some cases, more substantial securing[8] work was undertaken, mostly through the addition of adhesive anchors. However, securing work was only done in 56 (15%) cases. Therefore in total only 91 (25%) buildings had either visible shoring (49) and/or securing (56) work done on them following 4 September 2010.

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[7] The term 'shoring' refers to the addition of bracing members (usually braced to the ground or an adjacent building) to stabilise building elements from damage in subsequent aftershocks.

[8] The term 'securing' is used here to refer to temporary securing measures such as the addition of anchors to connect wall elevations to roof and floor diaphragms for temporary securing, or the addition of steel straps to hold cracked building corners together.

## 4.10 Directionality effects

A distinct correlation between building orientation and damage levels was observed, which is a characteristic referred to as 'directionality'. This directionality information is still being processed and is not reported herein.

## 4.11 Building demolition data

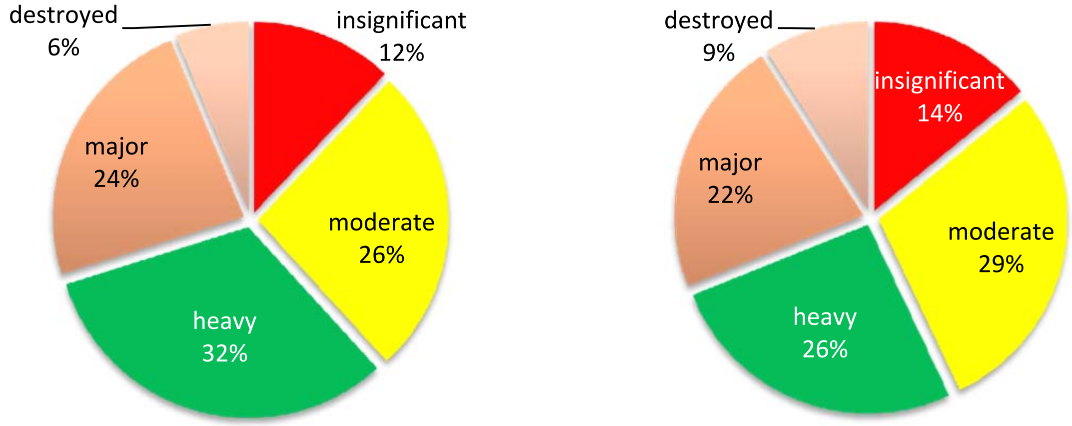
Figure 4.1(d) shows the state of buildings based upon demolition statistics published as at 4 October 2011. It is evident that over half (216, 58%) of the URM buildings included in the survey have already been demolished and that an additional 35 (10%) URM buildings are on the active demolition list (and may have already been demolished). 79 (21%) of the buildings appear to not be on the demolition list and are classified as 'standing', indicating that these building have currently been retained. See also section 4.11.1 for a critique on the condition of heritage listed buildings.

### 4.11.1 Condition of protected and heritage buildings

Of the 370 surveyed URM buildings, 192 (52%) buildings were either heritage buildings registered with the New Zealand Historic Places Trust (NZHPT) and/or were recognised as a protected building by CCC (note that of the two buildings for which no damage data was acquired, one building was 'Heritage and protected', resulting in 191 data entries). The damage data reported in Table 4.8 was further analysed to identify the assessed level of damage for those buildings that were either heritage listed or were protected by CCC, compared with those buildings that had no special classification. As shown in Table 4.11 and Figure 4.7 there was little difference between the damage distributions for the two groups of buildings.

**Table 4.11 Damage classifications for heritage and non-heritage buildings**

| Damage level | Heritage & protected | | Neither Heritage nor protected | |
|---|---|---|---|---|
| | No. of buildings | % of buildings | No. of buildings | % of buildings |
| insignificant | 23 | 12% | 25 | 14% |
| moderate | 50 | 26% | 51 | 29% |
| Heavy | 61 | 32% | 46 | 26% |
| Major | 45 | 24% | 39 | 22% |
| destroyed | 12 | 6% | 16 | 9% |
| **Total** | **191** | **100%** | **177** | **100%** |




(a) Heritage and protected buildings (b) Neither heritage nor protected buildings

**Figure 4.7 Damage classifications for heritage and non-heritage buildings**

Interestingly, Table 4.12 shows that 52 (27%) heritage and protected buildings remain standing whereas 28 (16%) of other buildings remain standing, and of the buildings that remain standing 52 (65% of 80) are heritage listed. Hence a higher proportion of heritage protected URM buildings have been retained, as would be expected both because these buildings are likely to have attracted a greater extent of earthquake strengthening, and because additional effort to avoid the demolition of heritage listed buildings would be expected.

**Table 4.12 Demolished and retained buildings**

| | **Heritage listed** | **Other** | **Total** |
|---|---|---|---|
| Demolished (incl. scheduled) | 128 (67% of 192) | 130 (73% of 178) | 258 (70%) |
| Standing | 52 (27% of 192) | 28 (16% of 178) | 80 (22%) |
| Unknown | 12 (6% of 192) | 20 (11% of 178) | 32 (8%) |
| **Total** | **192 (52% of 370)** | **178 (48%)** | **370 (100%)** |

# Section 5:

# The Performance of Earthquake Strengthening Techniques

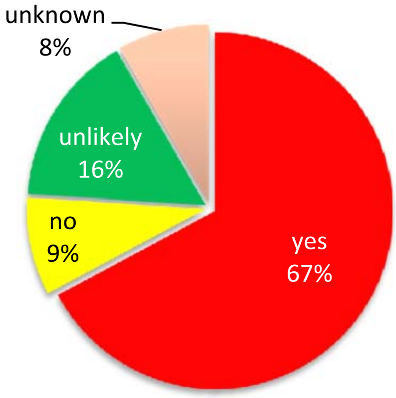
This section extends the information presented in Section 4, with details specifically provided regarding the performance in the 22 February 2011 Christchurch earthquake of various earthquake strengthening techniques installed within URM buildings located in the Christchurch CBD. The section begins with details regarding the distribution of earthquake strengthening improvements installed in heritage and protected URM buildings compared with their non-heritage equivalent, followed by the observed performance in the 22 February 2011 Christchurch earthquake of various earthquake strengthening techniques.

## 5.1 Earthquake strengthening of heritage buildings

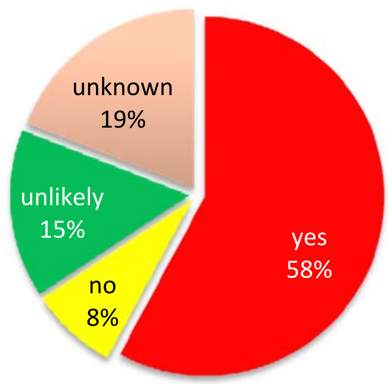
For the 370 buildings that were surveyed, an analysis was performed to identify the proportion of heritage and protected URM buildings that had received some form of earthquake strengthening, compared with the corresponding proportion for buildings that were not heritage protected. This information is reported in Table 5.1 and Figure 5.1, where it can be identified that the two classes of buildings had received comparable levels of earthquake strengthening. However, as might be expected, a greater proportion (67%) of heritage and protected buildings had received identifiable earthquake strengthening than for the non-heritage equivalent (58%), representing a comparative increase in the proportion of earthquake strengthening of 16% (67/58 = 1.155).

**Table 5.1 Distribution of earthquake strengthening for heritage and non-heritage URM buildings**

| Presence of earthquake strengthening | Heritage & protected buildings | | Neither Heritage nor protected buildings | | Combined dataset | |
|---|---|---|---|---|---|---|
| | No. of buildings | % of buildings | No. of buildings | % of buildings | No. of buildings | % of buildings |
| yes | 129 | 67% | 102 | 58% | 231 | 63% |
| no | 17 | 9% | 14 | 8% | 31 | 8% |
| unlikely | 30 | 16% | 27 | 15% | 57 | 15% |
| unknown | 15 | 8% | 34 | 19% | 49 | 14% |
| **Total** | **191** | **100%** | **177** | **100%** | **368** | **100%** |




(a) Heritage and protected buildings




(b) Non-heritage buildings

**Figure 5.1 Distribution of earthquake strengthening implementation**

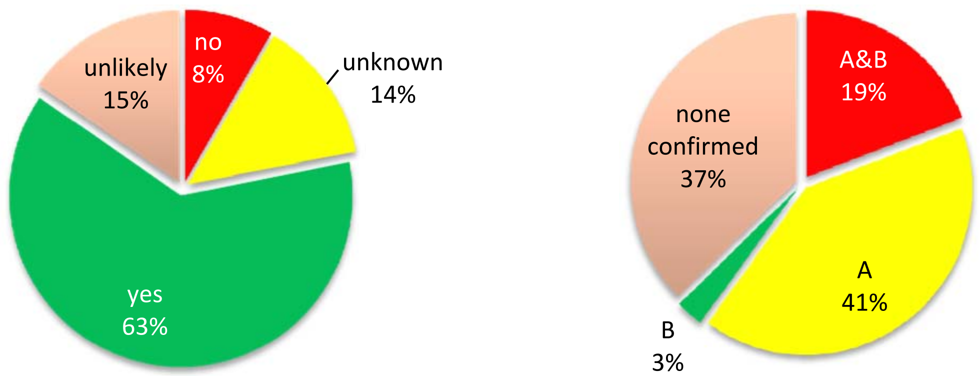
## 5.2 Number of earthquake strengthened URM buildings

As outlined in section 3.11, the various forms of earthquake strengthening of URM buildings were categorised into three types, being parapet restraints (see section 3.11.1), Type A seismic retrofits (see 3.11.2), and Type B seismic retrofits (see section 3.11.3). The distribution of different Type A and Type B earthquake strengthening improvements is outlined in Table 5.2, with details of the earthquake performance of URM buildings having parapet restraints discussed in section 5.3.

**Table 5.2 Distribution of earthquake strengthening types**

| Type of earthquake strengthening | No. of buildings | % of buildings |
|---|---|---|
| A&B | 82 | 22% |
| A | 149 | 40% |
| none confirmed | 139 | 38% |
| **Total** | **370** | **100%** |

As can be seen in Table 5.2, 231 (62%) of all URM buildings in the Christchurch CBD had some form of earthquake strengthening installed at the time of the 22 February 2011 Christchurch earthquake (in addition to parapet restraints possibly being installed also), with 82 (22%) buildings identified as having one or more Type B earthquake strengthening methods installed. This data is shown graphically in Figure 5.2.




(a) Proportion of earthquake strengthened buildings

(b) Identified type earthquake strengthening

**Figure 5.2 Distribution of installed earthquake strengthening types**

## 5.3 Parapet restraints

A total of 435 records of parapets were associated with the surveyed buildings, with some buildings having multiple parapets, such as those on street corners or for end-of-row or stand-alone buildings. As shown in Table 5.3, of these 435 parapets only 149 (34%) parapets could be positively identified as having parapet restraints installed. Unfortunately, it was not possible to definitively identify a sufficient sample size of specific types of parapet restraint systems from building inspections.

**Table 5.3 Distribution of parapet restraints**

| | No. of cases | % of parapets |
|---|---|---|
| restrained | 149 | 34% |
| unrestrained | 89 | 21% |
| unknown | 197 | 45% |
| **Total** | **435** | **100%** |

As expected (see Table 5.4 and Figure 5.3), restrained parapets performed significantly better than parapets having no restraint, with 75 (84%) unrestrained parapets suffering full or partial collapse while only 65 (44%) restrained parapets suffered similar damage. Furthermore, 71 (48%) restrained parapets suffered no or moderate damage while only 12 (13%) unrestrained parapets achieved such good response, such that 86% (71/83) of those parapets that performed satisfactorily were restrained.

**Table 5.4 Damage sustained by restrained and unrestrained parapets**

| Damage classification | Restrained Parapet | Unrestrained Parapet |
|---|---|---|
| none | 35 (23%) | 5 (6%) |
| moderate | 36 (24%) | 7 (8%) |
| heavy | 13 (9%) | 2 (2%) |
| partial collapse | 29 (20%) | 25 (28%) |
| full collapse | 36 (24%) | 50 (56%) |
| **Total** | **149 (100%)** | **89 (100%)** |

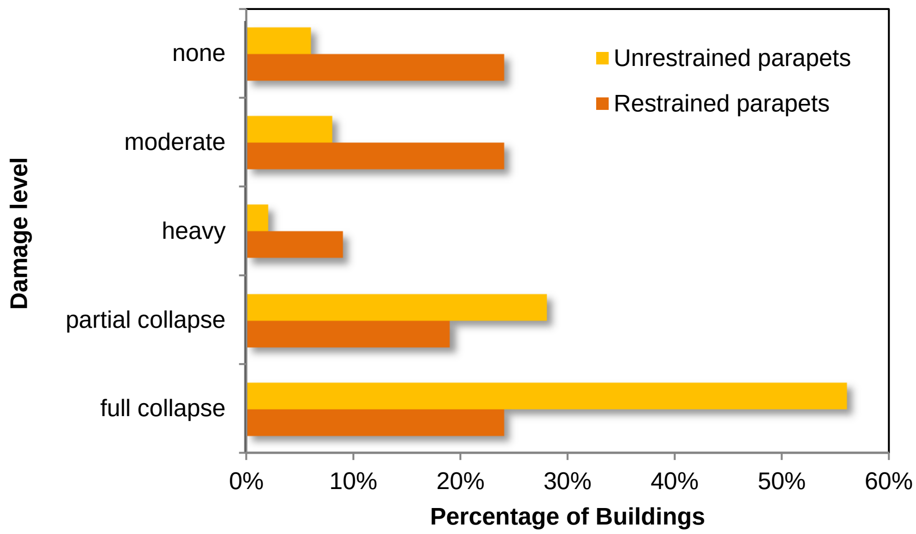



**Figure 5.3 Performance of unrestrained and restrained parapets**

Overall it may be concluded that unrestrained parapets were twice as likely to collapse as were restrained parapets. It would seem that this is a disappointing finding as it would have been sensible to assume that the majority of restrained parapets would have performed satisfactorily. This finding suggests that further investigation is required to better understand why parapet restraints were not more uniformly successful in preventing damage.

## 5.4 Type A earthquake strengthening techniques

As detailed in section 3.11.2, Type A retrofits include gable restraints, wall-to-diaphragm anchorage, and roof and floor diaphragm improvement. Earthquake performance details associated with this class of earthquake strengthening is reported below. As shown in Table 5.2 and also Table 5.5, 231 (62%) of all URM buildings in the Christchurch CBD had some form of Type A earthquake strengthening installed.

**Table 5.5 Distribution of buildings having Type A earthquake strengthening**

| Type A retrofit installed | No of buildings | % of buildings |
|---|---|---|
| yes | 231 | 62% |
| no | 26 | 7% |
| unlikely | 57 | 15% |
| unknown | 56 | 15% |
| **Total** | **370** | **100%** |

The various forms of Type A earthquake strengthening that were installed are reported in Table 5.6, showing that the use of through ties was the most commonly encountered form of Type A earthquake strengthening. In addition, 52 URM buildings (14% of 370 total population) had diaphragm stiffening installed, with 18 buildings being identified as having both floors and roof diaphragm stiffening.

**Table 5.6 Forms of Type A earthquake securing**

| Form of Type A retrofit | No of buildings | % of buildings |
|---|---|---|
| through ties | 162 | 70% |
| adhesive anchors | 39 | 17% |
| other | 2 | 1% |
| original | 6 | 3% |
| unknown | 22 | 10% |
| **Total** | **231** | **100%** |

Of the 231 buildings where a Type A retrofit was implemented, 162 (70%) of these retrofits used ‘through tie’ anchors. However, as ‘through ties’ are the most easily identified Type A earthquake strengthening method, this would have influenced the number of identified installations. The next most common form of Type A earthquake strengthening was adhesive anchors, with 39 (17%) buildings identified as using this type of securing. Only 6 (3%) buildings were identified as exclusively having original wall-diaphragm connections (which technically is not part of earthquake strengthening improvement). This low percentage of observed cases of original earthquake resistant construction is likely to be due to both the difficultly of identifying this connection form and the rarity of its occurrence. Further details of the performance of Type A earthquake strengthening techniques are reported below.

### 5.4.1 Gable end wall restraints

A total of 185 wall elevations having gables were identified in the survey. Table 5.7 shows the distribution of gable restraints for these gable ended walls, indicating that 129 (70%) gable end walls had gable restraints installed.

**Table 5.7 Prevalence of restraints for gable end walls**

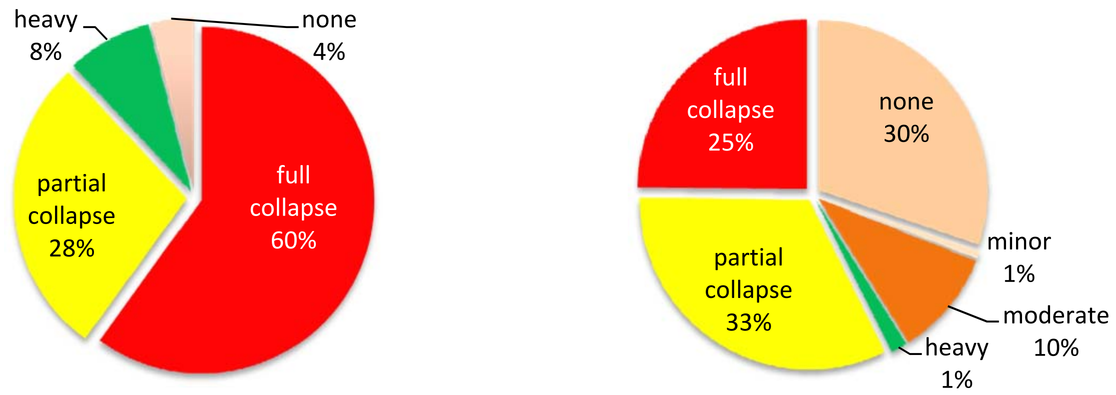
| Type of restraint | Number (%) |
|---|---|
| no restraint present | 25 (14%) |
| gable restraints identified | 129 (70%) |
| unknown | 31 (17%) |
| **Total** | **185 (100%)** |

Table 5.8 shows the distribution of the type of restraints used to secure the gable elevation from out-of-plane collapse, with 108 (84%) of the identified gable restraints being 'through ties' (see section 3.11.2.1). It should be noted that 'through ties' are more readily identifiable than many other restraint types, especially adhesive anchors, and so the reported distribution is likely to under-count securing types such as adhesive anchor systems that are less readily identified.

**Table 5.8 Identified types of gable restraints**

| Restraint types | % of gables (out of 129 gables) |
|---|---|
| original | 14 (10%) |
| through ties | 108 (84%) |
| adhesive anchors | 5 (4%) |
| through ties + concrete beam | 1 (1%) |
| other | 1 (1%) |
| **Total** | **129 (100%)** |

As can be seen in Figure 5.4, restrained gables performed better than for situations where no gable restraint was identified. However, 74 (57%) restrained gables suffered partial or full collapse.




(a) Damage levels to unrestrained gables, all elevations

(b) Damage levels to restrained gables, all elevations

**Figure 5.4 Gable damage levels**

As discussed above, Table 5.8 shows that there were 108 gable elevations where ‘through ties’ were used to provide restraint. This data population allowed the specific performance of through ties to be further analysed, as shown in Table 5.9 and Figure 5.5. From this analysis it may be determined that this securing technique resulted in mixed success, with 58 (54%) gable elevations suffering in excess of moderate damage. Further correlation between damage levels and the spacing of through tie anchors would assist in determining future recommendations for earthquake strengthening procedures, but is beyond the scope of the study reported here. Nevertheless, the finding that approximately half of all restrained gables sustained either heavy damage or collapse suggests that further investigation of appropriate procedures for the earthquake protection of unreinforced masonry gable end walls is merited.

**Table 5.9 Damage levels for gables restrained using through ties**

| Damage level for gables restrained with through ties | No. of buildings | % of buildings |
|---|---|---|
| none | 39 | 36% |
| minor | 0 | 0% |
| moderate | 11 | 10% |
| heavy | 2 | 2% |
| partial collapse | 36 | 33% |
| full collapse | 20 | 19% |
| **Total** | **108** | **100%** |

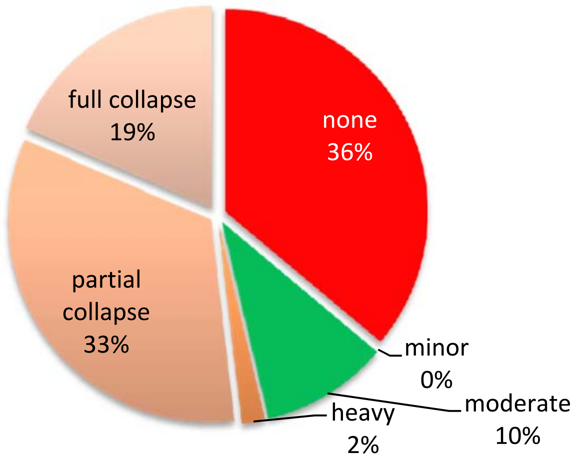



**Figure 5.5 Damage distribution for gables restrained using through ties**

### 5.4.2 Roof diaphragm improvement

Roof diaphragm improvement was identified in 37 (17%) URM buildings where Type A earthquake strengthening was implemented (see Table 5.10). The most common roof diaphragm improvement type was steel bracing (19 cases), followed by steel brace frames (12 cases) and plywood overlay (6 cases). However, for 162 (70%) earthquake strengthened buildings it was not possible to confirm whether roof diaphragm improvements were implemented.

**Table 5.10 Distribution of roof diaphragm improvements**

| Roof diaphragm stiffening | No. of buildings | % of buildings |
|---|---|---|
| plywood overlay | 6 | 3% |
| steel bracing | 19 | 8% |
| steel brace frame | 12 | 5% |
| other | 1 | 0% |
| none | 31 | 13% |
| unknown | 162 | 70% |
| **Total** | **231** | **100%** |

### 5.4.3 Floor diaphragm improvement

Floor diaphragm improvement was confirmed in only 32 (14%) buildings where Type A earthquake strengthening was implemented, and was not able to be confirmed in 187 (81%) URM buildings (see Table 5.11). For those buildings where floor diaphragm improvement had been implemented, plywood overlay was the most common stiffening type, with 20 (9%) cases identified. Four (2%) cases each of steel bracing, steel brace frames and the addition of concrete were also identified. Of the four concrete floor improvements identified, three were in-situ concrete floors over the full floor area to replace the original timber floor diaphragm and one floor was a concrete overlay above the original timber floor, with supplementary steel beams added to support the added weight of the concrete overlay.

**Table 5.11 Distribution of floor diaphragm improvements**

| Floor diaphragm stiffening | No. of buildings | % of buildings |
|---|---|---|
| plywood overlay | 20 | 9% |
| steel bracing | 4 | 2% |
| steel brace frame | 4 | 2% |
| concrete | 4 | 2% |
| none | 12 | 5% |
| unknown | 187 | 80% |
| **Total** | **231** | **100%** |

## 5.5 Type B earthquake strengthening techniques

As shown in Table 5.2 and Table 5.12, 82 (22%) URM buildings in the Christchurch CBD had various Type B earthquake strengthening techniques installed.

**Table 5.12 Distribution of buildings having Type B earthquake strengthening**

| Type B retrofit installed | No of buildings | % of buildings |
|---|---|---|
| yes | 82 | 22% |
| no | 63 | 17% |
| unlikely | 120 | 32% |
| unknown | 105 | 28% |
| **Total** | **370** | **100%** |

Within these 82 URM building having Type B earthquake strengthening, there were some cases where multiple Type B strengthening techniques were used, such that overall a total of 109 Type B installations were surveyed, as reported in Table 5.13. The most common Type B earthquake strengthening technique surveyed was the addition of steel moment frames, followed closely by concrete moment frames (see Table 5.13).

**Table 5.13 Distribution of Type B strengthening techniques encountered**

| Strengthening technique | No. of buildings | % of buildings |
|---|---|---|
| steel moment frames | 24 | 22% |
| steel brace frames | 14 | 13% |
| strong-backs - internal | 14 | 13% |
| strong-backs - external | 4 | 4% |
| concrete moment frames | 22 | 20% |
| addition of cross walls | 13 | 11% |
| shotcrete | 10 | 9% |
| FRP | 1 | 1% |
| post tensioning | 2 | 2% |
| other | 5 | 5% |
| **Total** | **109** | **100%** |

The overall building damage levels for 97 (89%) of the 109 identified cases of Type B earthquake strengthening were considered, with the damage distribution shown in Table 5.14 and Figure 5.6. The Type B strengthening methods of Fibre Reinforced Polymer (FRP), post-tensioning, and external strong-backs were not included in Table 5.14 as the number of surveyed cases of implementation was small. Note that buildings having more than one form of Type B seismic strengthening (eg. steel moment frames and internal strong backs) are represented twice in Table 5.14 and Figure 5.6 such that the 97 entries do not represent 97 different buildings.

**Table 5.14 Damage level for various Type B earthquake strengthening techniques**

| Method of Type B strengthening | Destroyed | Major | Heavy | Moderate | Insignif. | Total |
|---|---|---|---|---|---|---|
| Shotcrete | 0 (0%) | 0 (0%) | 0 (0%) | 5 (50%) | 5 (50%) | 10 (100%) |
| Strong backs - Internal | 0 (0%) | 2 (14%) | 1 (7%) | 8 (57%) | 3 (22%) | 14 (100%) |
| Steel moment frames | 0 (0%) | 1 (4%) | 7 (29%) | 13 (54%) | 3 (13%) | 24 (100%) |
| Addition of cross walls | 0 (0%) | 0 (0%) | 5 (38%) | 3 (24%) | 5 (38%) | 13 (100%) |
| Concrete moment frames | 1 (5%) | 3 (14%) | 6 (27%) | 7 (32%) | 5 (23%) | 22 (100%) |
| Steel brace frames | 0 (0%) | 3 (21%) | 4 (29%) | 5 (36%) | 2 (14%) | 14 (100%) |
| Total | 1 (1% of 97) | 9 (9% of 97) | 23 (24% of 97) | 41 (42% of 97) | 23 (24% of 97) | 97 (100% of 97) |

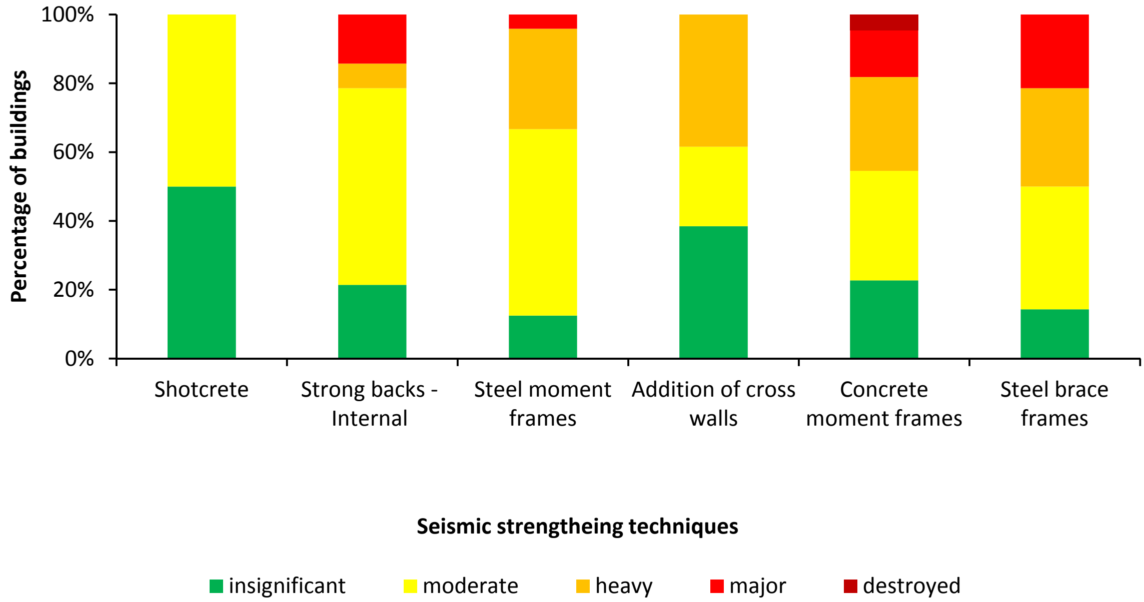



**Figure 5.6 Building damage distribution for various Type B earthquake strengthening techniques**

As might be expected, the data presented in Table 5.14 and Figure 5.6 shows that the two Type B earthquake strengthening methods that resulted in the least damage were shotcrete and the addition of cross walls. The two methods are somewhat analogous as the former involves the application of new reinforced concrete walls adhered to the exterior of existing URM walls and the latter involves the installation of new walls. The data also shows that the damage statistics for buildings having internal strong backs, steel moment frames and concrete moment frames was comparable.

## 5.6 Comparison between earthquake strengthening schemes and overall building damage

In this section the building damage data for 368 URM buildings in the Christchurch CBD is correlated against earthquake strengthening schemes, recalling that building damage information was not available for two URM buildings. As shown in Table 5.15, URM buildings having no earthquake strengthening suffered greater damage than those buildings that had received some form of earthquake strengthening. Of the 31 URM building confirmed to have no earthquake strengthening, 30 (97%) suffered heavy, major or severe damage (see Table 5.15 and Figure 5.7). Similarly, of the 149 buildings that had a Type A earthquake strengthening scheme installed, 104 (70%) suffered heavy, major, or severe damage whereas for combined Type A&B strengthening only 29 (35%) buildings suffered this level of damage.

**Table 5.15 Damage level for different types of earthquake strengthening**

| Damage level | Seismic strengthening type | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | No retrofit | | Type A | | Type A&B | | Unknown | |
| Insignificant | 0 | 0% | 10 | 7% | 19 | 23% | 19 | 18% |
| Moderate | 1 | 3% | 35 | 23% | 34 | 41% | 31 | 29% |
| Heavy | 16 | 52% | 48 | 32% | 20 | 24% | 23 | 22% |
| Major | 9 | 29% | 42 | 28% | 8 | 10% | 25 | 24% |
| Destroyed | 5 | 16% | 14 | 9% | 1 | 1% | 8 | 8% |
| **Heavy, Major and Destroyed combined** | **30 of 31** | **97%** | **104 of 149** | **70%** | **29 of 82** | **35%** | **56 of 106** | **53%** |

From this data it is clear that Type B earthquake strengthening was more successful at minimising building damage than were Type A earthquake strengthening improvements. This observation is to be expected, recognising that the aim of Type B earthquake strengthening improvements is to increase the global earthquake capacity of the building, and consequently reduce the expected overall building damage level.

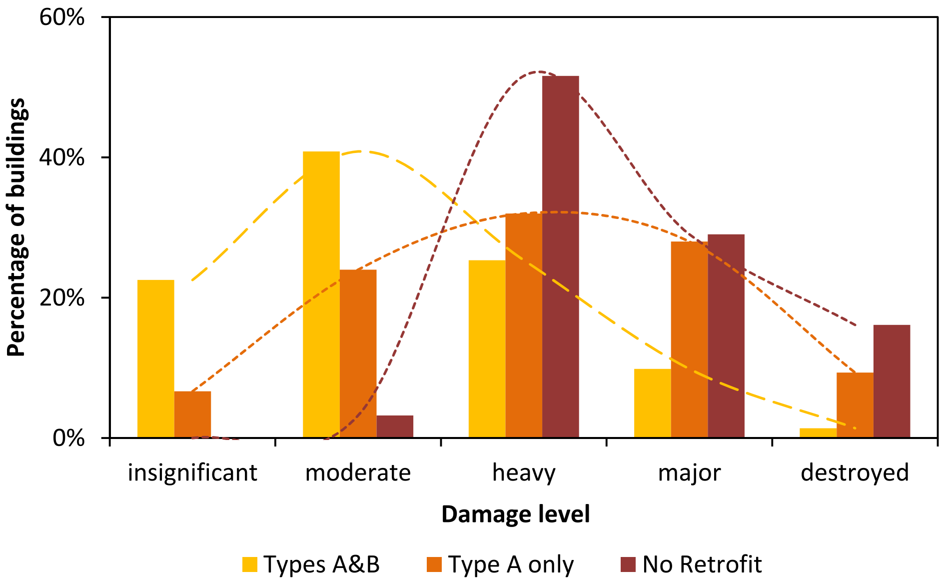



**Figure 5.7 Plot of damage level against seismic strengthening types**

# Section 6:

# Influence of earthquake strengthening level on observed damage and assessed hazard

In this section the data from Section 4 and Section 5 are extended to investigate the relative performance of URM buildings in the Christchurch CBD that had received various levels of earthquake strengthening. The %NBS parameter is used throughout to describe the level of earthquake strengthening, with all calculations performed using a zone factor for Christchurch of Z = 0.22, which was the zone factor in place at the time of the 22 February 2011 earthquake. The section concludes with an analysis of the hazard to building occupants and to passers-by due to the observed extent of building damage.

## 6.1 Comparison of %NBS and assessed damage level

Earthquake strengthening levels in terms of %NBS were identified for 94 (26% of the 368 total) URM buildings, either by consulting CCC records, by personal communication with building owners, engineers and heritage personnel, or in cases where sufficient details about the building were known was based upon estimation. The distribution of %NBS data is reproduced in Table 6.1 where it is shown that 61 (65% of 94) URM buildings had been earthquake strengthened to at least 67%NBS and a further 18 (19%) URM buildings had been earthquake strengthened to at least 34%NBS. 15 (16%) buildings had been strengthened to less than 33%NBS, and 31 (8% of the 368 total) URM buildings were positively confirmed to have received no earthquake strengthening. Further investigation will facilitate the identification of whether the remaining 243 (=368-94-31) URM buildings in the Christchurch CBD for which damage data is available had received any form of earthquake strengthening. However, this expanded investigation was not undertaken in order to expedite the release of this report.

**Table 6.1 Distribution of %NBS classifications for 94 earthquake strengthened URM buildings**

| NBS Retrofit level | No. of buildings | % of buildings |
|---|---|---|
| %NBS < 33 | 15 | 16% |
| 33 ≤ %NBS < 67 | 18 | 19% |
| 67 ≤ %NBS < 100 | 50 | 53% |
| %NBS ≥ 100 | 11 | 12% |
| **Total** | **94** | **100%** |

The performance of these 94 earthquake strengthened buildings and 31 unstrengthened buildings was analysed by determining the damage distribution for each category of %NBS as shown in Table 6.2 and Figure 6.1.

**Table 6.2 Damage levels for different %NBS categories**

| | **%NBS ≥ 100** | | **67 ≤ %NBS < 100** | | **33 ≤ %NBS < 67** | | **%NBS < 33** | | **No retrofit** | |
|---|---|---|---|---|---|---|---|---|---|---|
| Insignificant 1 - 10% | 8 | 73% | 10 | 20% | 1 | 6% | 1 | 7% | 0 | 0% |
| Moderate 10 - 30% | 3 | 27% | 28 | 56% | 4 | 22% | 5 | 33% | 1 | 3% |
| Heavy 30 - 60% | 0 | 0% | 10 | 20% | 9 | 50% | 5 | 33% | 16 | 52% |
| Major 60 - 100% | 0 | 0% | 2 | 4% | 4 | 22% | 1 | 7% | 9 | 29% |
| Destroyed 100% | 0 | 0% | 0 | 0% | 0 | 0% | 3 | 20% | 5 | 16% |
| **Combined Heavy, Major and Destroyed** | **0 of 11** | **0%** | **12 of 50** | **24%** | **13 of 18** | **72%** | **9 of 15** | **60%** | **30 of 31** | **97%** |
| **Total** | **11** | | **50** | | **18** | | **15** | | **31** | |

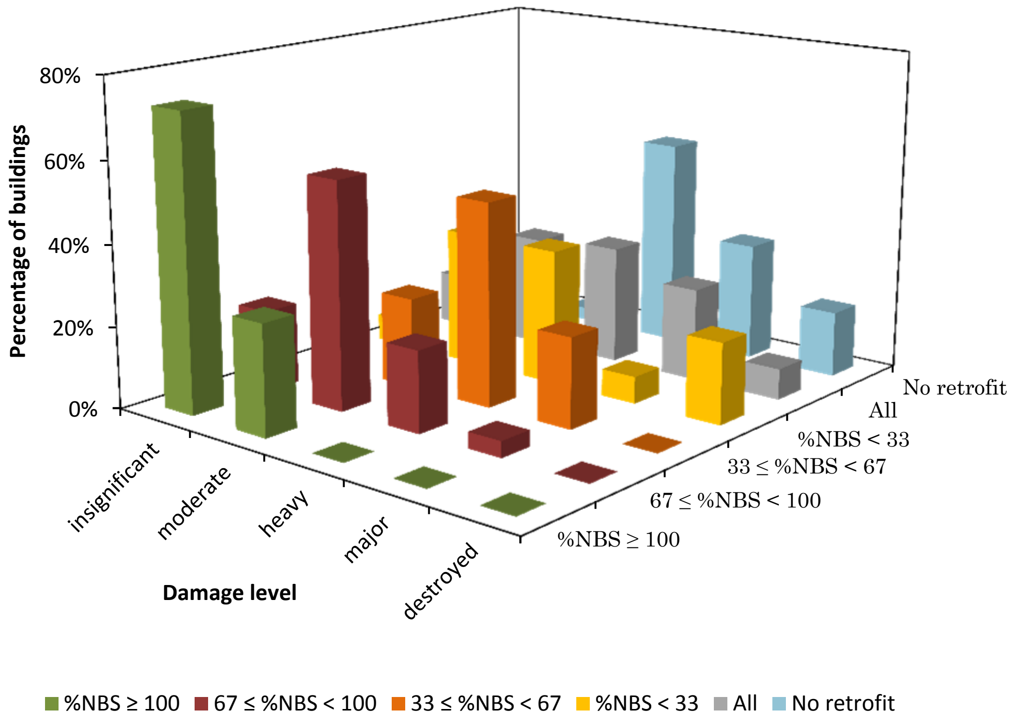



**Figure 6.1 Damage levels for different levels of %NBS earthquake strengthening**

The data plotted in Figure 6.1 is reproduced in a different format in Figure 6.2. From Table 6.2, Figure 6.1, and Figure 6.2 it can be determined that those URM buildings strengthened to 100%NBS performed well, that buildings strengthened to 67%NBS performed moderately well, but that buildings strengthened to less than 33%NBS collectively exhibited no significant improvement in performance when compared with buildings that had received no earthquake strengthening.

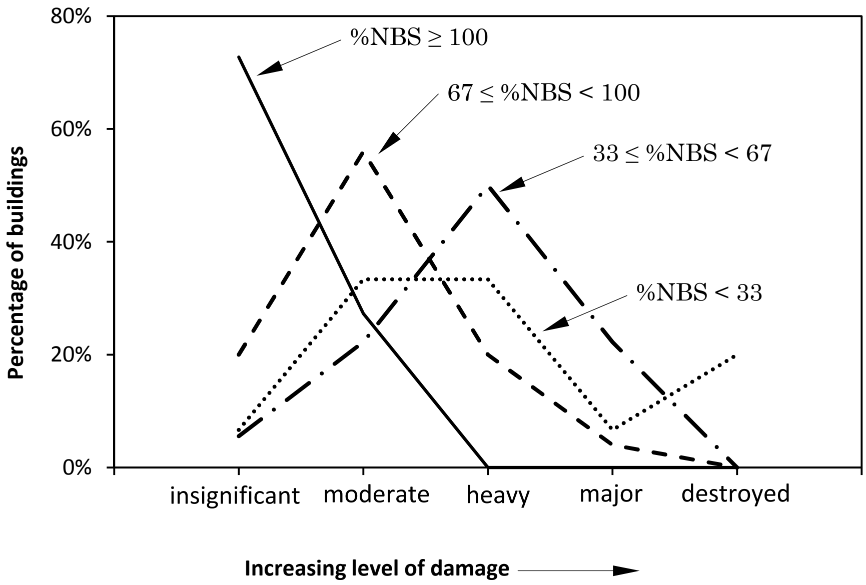



**Figure 6.2 Plot of overall building damage level for different levels of %NBS earthquake strengthening**

Figure 6.3, Figure 6.4 and Figure 6.5 show earthquake performance for buildings strengthened to increasing levels of %NBS, plotted against the performance of buildings that received no earthquake strengthening. As anticipated, the three plots show that an increase in the level of %NBS earthquake strengthening resulted in reduced levels of damage.

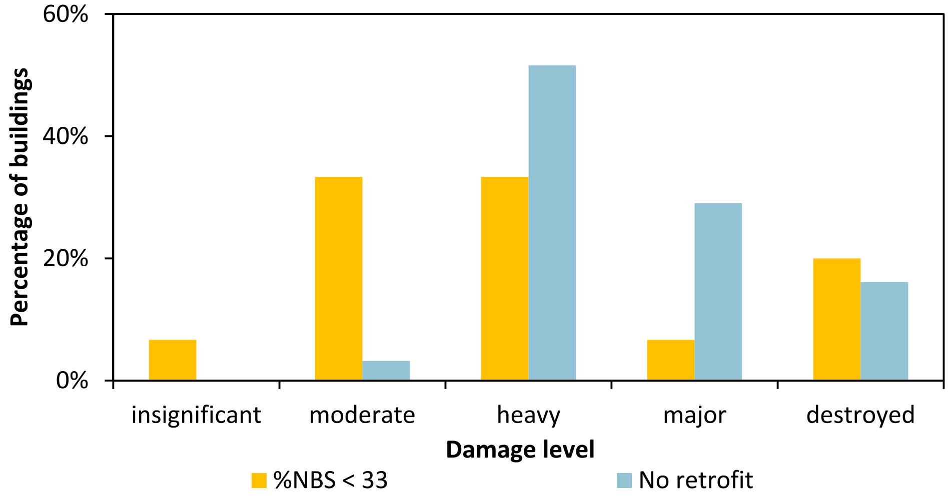



**Figure 6.3 Damage comparison between URM buildings strengthened to 33%NBS and no retrofit**

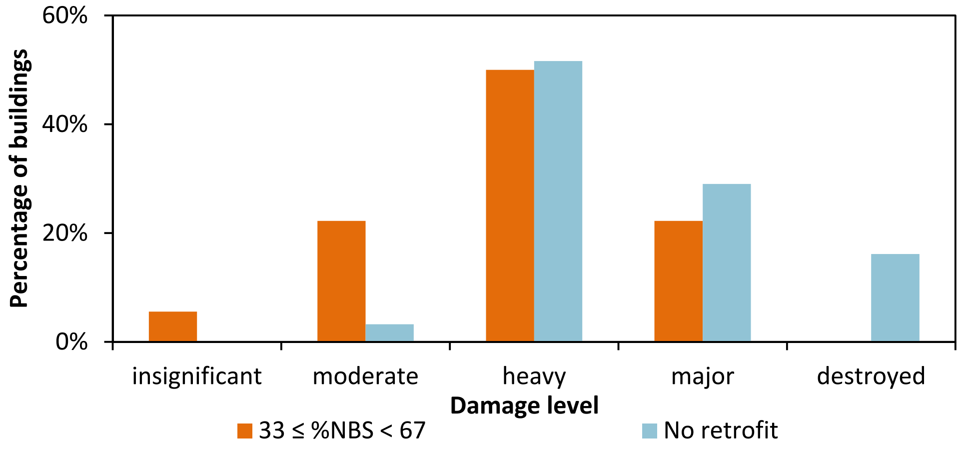



**Figure 6.4 Damage comparison between URM buildings strengthened to 33-67%NBS and no retrofit**

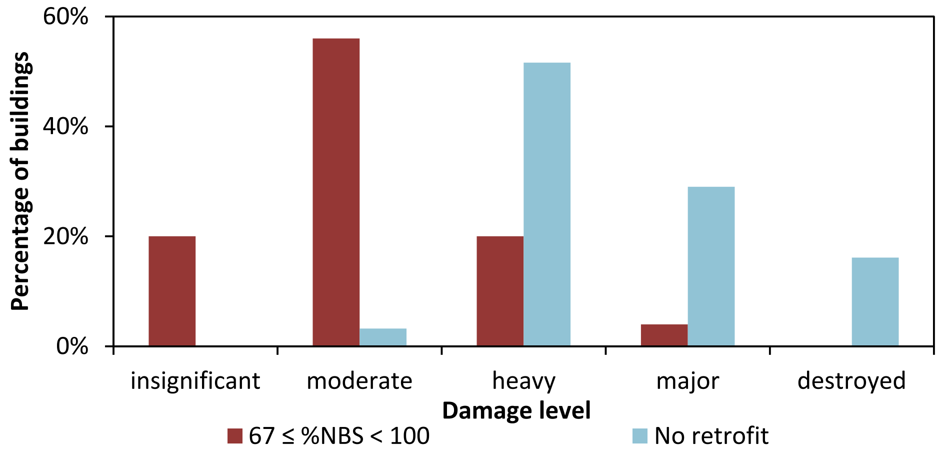



**Figure 6.5 Damage comparison between URM buildings strengthened to 67-100%NBS and no retrofit**

Figure 6.6 and Figure 6.7 extend the analysis reported above, by comparing damage levels for different levels of earthquake strengthening.

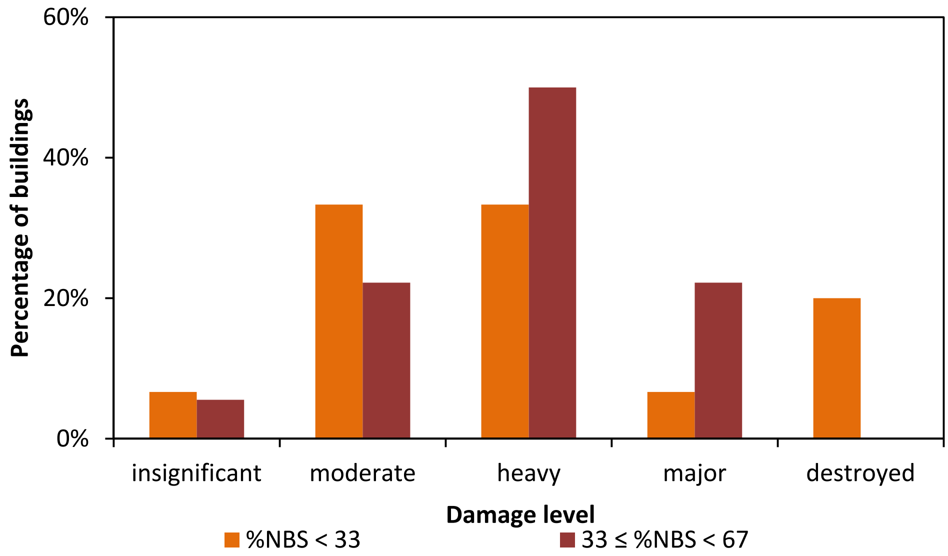



**Figure 6.6 Damage comparison between URM buildings strengthened to 0-33%NBS or to 33-67%NBS**

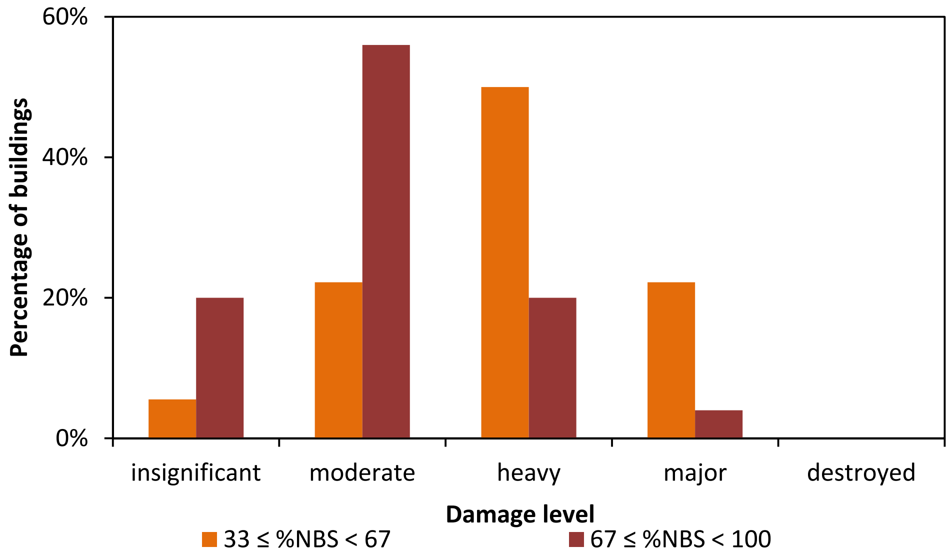



**Figure 6.7 Damage comparison between URM buildings strengthened to 33-67%NBS or 67-100%NBS**

**Table 6.3 Damage index for different levels of earthquake strengthening**

| %NBS Retrofit Level | Damage "index" |
|---|---|
| no retrofit | 63 |
| %NBS < 33 | 47 |
| 33 ≥ %NBS < 67 | 45 |
| 67 ≥ %NBS < 100 | 24 |
| %NBS ≥ 100 | 9 |
| all buildings | 45 |

In Table 6.3 a damage index is presented for each level of earthquake strengthening. This index has been calculated as the midpoint % damage as reported in Table 6.2 for each damage level, multiplied by the number of surveyed building at that damage level proportional to the total number of buildings strengthened to that level. Hence for buildings strengthened to less than 33%NBS this calculation is:

$$\text{DL for }\%\text{NBS} \leq 33 = 5\% \times \frac{1}{15} + 20\% \times \frac{5}{15} + 45\% \times \frac{5}{15} + 80\% \times \frac{1}{15} + 100\% \times \frac{3}{15} = 47$$

### 6.1.1 Damage interpretations

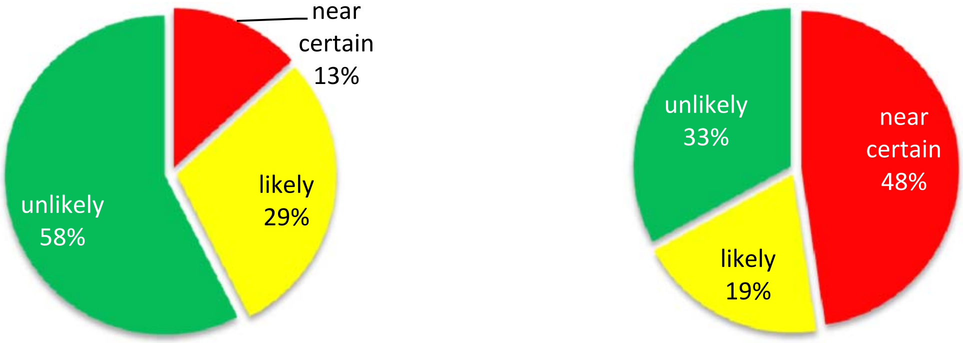
Based upon the data presented above the following interpretations can be made:

1. From Figure 6.3 it can be determined that URM buildings that were strengthened to less than 33%NBS performed in an approximately similar manner to unstrengthened URM buildings. However, this level of strengthening did result in a significant reduction from major damage to moderate damage. This damage reduction is reflected in the damage indices for these two strength categories, with a damage index of 63 calculated for unstrengthened URM buildings and a damage index of 47 calculated for URM building strengthened to 0-33%NBS.
2. From Figure 6.4 it can be determined that URM building strengthened to 33-67%NBS avoided being destroyed (100% damage), but that otherwise their performance was not greatly better than for unstrengthened buildings. Figure 6.6 provides further clarification of this matter, as does the category ‘Combined Heavy, Major and Destroyed’ in Table 6.2, which reports 72% damage for 33-67%NBS and 60% damage for 0-33%NBS, representing an <u>increase</u> in damage of 20% (=72%/60%) for the 33-67%NBS earthquake strengthening class. Finally, the comparable damage indices (47 vs 45) reported in Table 6.3 for these two strength classes again indicates minimal overall reduction in damage when compared to the 0-33%NBS strength class.
3. Figure 6.5 and Figure 6.7 both show that URM buildings strengthened to 67-100%NBS performed much better than both URM buildings having no strengthening and URM buildings strengthened to lower levels of earthquake resistance. This significant improvement in performance is also reflected in the

reduced damage index reported in Table 6.3 for this strengthening level. This finding is consistent with Recommendation 4 in Report No. 1.

## 6.2 Risk to building occupants and passers-by

From survey observations and a critique of photographs of damage taken for each building, a hazard analysis was performed to consider the risk posed to people if they had hypothetically been inside or directly outside (such as on the footpath) each URM building during the earthquake, with the latter scenario referred to here as a 'passer-by'. It is recognised that this hazard analysis hazard (see section 3.10 for description of different levels of risk) is subjective, but the result is reported in an attempt to quantify the hazard posed by building damage and falling masonry debris. As can be seen from Figure 6.8, the results suggest that it is generally safer to be inside a URM building during an earthquake than to be directly outside the building.




(a) Risk to building occupants (b) Risk to public space occupants

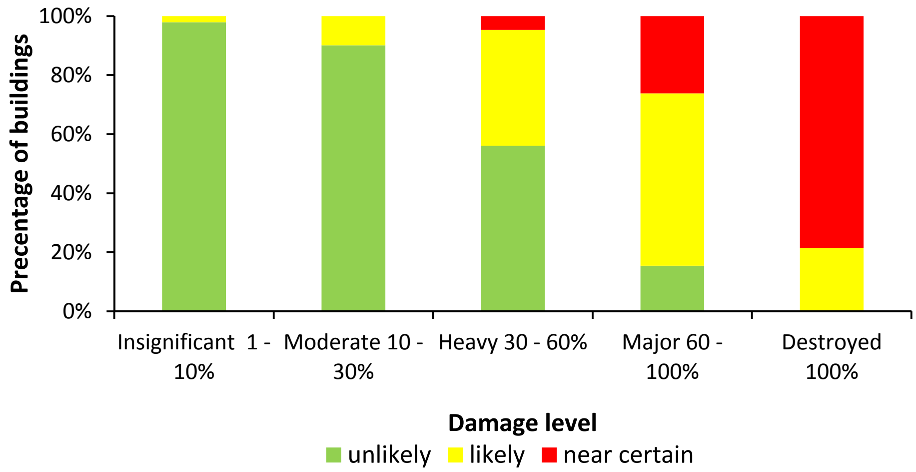
**Figure 6.8 Risk of fatality or injury to building occupants and public space occupants**

When considered in greater detail, it was identified that buildings which posed a near certain risk of fatality or injury to their occupants also posed a comparable risk for anyone occupying the public space directly adjacent to the building. However, buildings which posed a risk to passers-by were not necessarily a risk to the building's occupants. This is because walls are more likely to collapse outwards, due to restraint provided by contact with roof and floor diaphragms for wall deformations directed towards the building interior. Interior falling debris is less likely to be due to falling masonry parapets, gables and walls as usually roofs, upper storey floors, and even interior partitioning provide some protection to the building occupants.

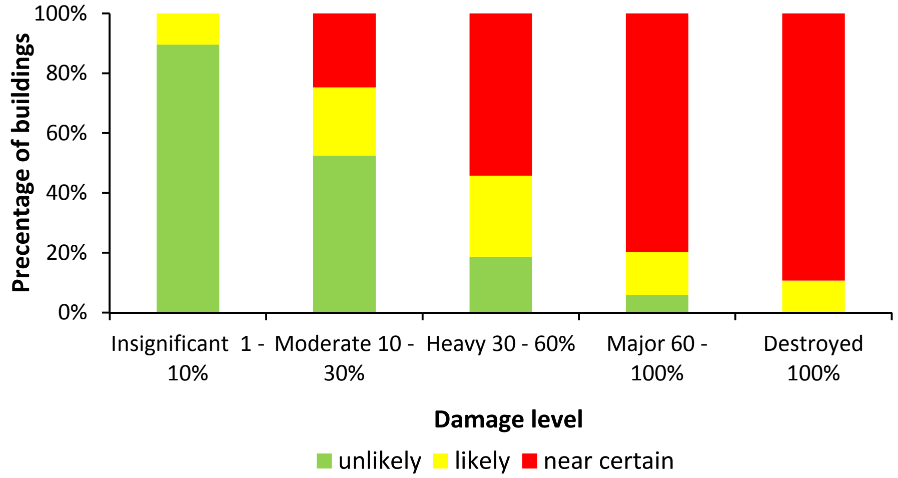
From this hazard analysis it is identified that the risk to the general public 'passer-by' and the risk to the building occupants differ, and that both hazard scenarios need to be considered. Directly outside the building the public are at greater risk due to falling masonry parapets, gables and walls.

### 6.2.1 Occupant and passer-by hazard for differing levels of building damage

The analysis described above was extended to consider the escalating hazard to building occupants and to passers-by for increased levels of building damage. Obviously, as the overall building damage level increases, so too does the risk of fatality or injury to the public and to the building occupant. However, as shown in Figure 6.9 the danger to the passer-by increases more significantly with increasing damage than does the risk to building occupants.




(a) Risk to building occupant for different building damage levels



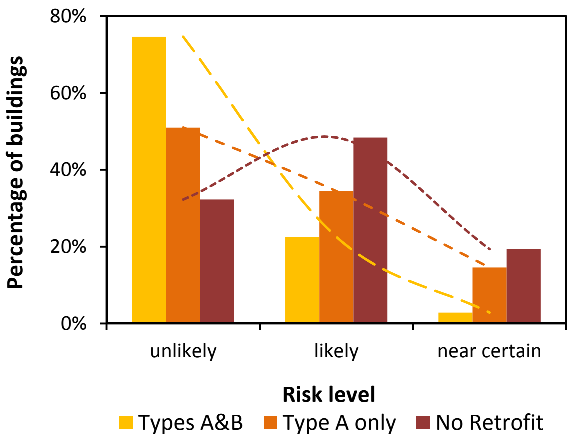

(b) Risk to passer-by for different damage levels

**Figure 6.9 Fatality and injury risk for different building damage levels**

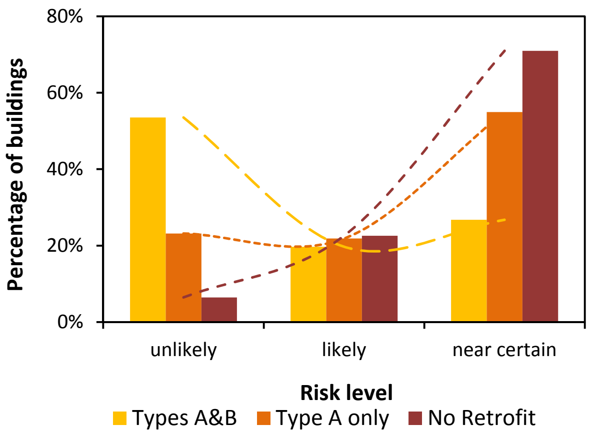
### 6.2.2 Occupant and passer-by hazard for differing levels of earthquake strengthening

The correlation between earthquake strengthening levels and the hazard to building occupants and to passers-by was assessed. As expected, and as shown in Figure 6.10(a), buildings that had undergone Type A + B earthquake strengthening posed significantly less risk to their occupants when compared to buildings that had undergone Type A earthquake strengthening only, with the greatest risk being for buildings that had received no earthquake strengthening.

In Figure 6.10(b) it is particularly evident that buildings that has undergone Type A + B seismic strengthening were much less likely to pose a risk to passers-by than were buildings having only Type A earthquake strengthening or buildings having no earthquake strengthening.




(a) Risk to building occupants




(b) Risk to public space occupants

**Figure 6.10 Plot of seismic strengthening level vs. risk to building occupants and public spaces**

### 6.2.3 Damage due to neighbouring buildings

Of the 370 buildings surveyed the majority (260, 70%) suffered no damage from neighbouring buildings, as shown in Table 6.4. However, the remainder (110, 30%) of the surveyed buildings sustained damage levels due to neighbouring buildings, ranging from minor to severe although nearly three-quarters of buildings sustained no damage or minor damage only. In a number of cases damage that otherwise would not have occurred to earthquake strengthened buildings was attributable to full or partial collapse or to falling debris from neighbouring buildings that had received no or little earthquake strengthening.

**Table 6.4 Damage level due to neighbouring building**

| Damage level | No. of buildings | % of buildings |
|---|---|---|
| Severe | 21 | 6% |
| Moderate | 42 | 11% |
| Minor | 47 | 13% |
| None | 260 | 70% |

# Section 7:

# Sample collection and in-situ testing

In conjunction with the building survey exercise reported earlier herein, two companion studies were undertaken that are briefly summarised in this section. The first exercise involved the collection of mortar and brick samples from damaged URM buildings, so that this data could be used to extend the existing material property database compiled by Ronald Lumantarna as part of his doctoral investigation. It is intended that the full database will soon be published, so that designers can use this information as a resource when undertaking seismic assessment and improvement projects on URM buildings.

The second reported study was funded by the US National Science Foundation as part of their RAPID grant fund, and involved a research collaboration between the University of Minnesota and the University of Auckland, led by Professor Arturo Schultz. This study addressed the pull out strength of adhesive anchors, recognising that many such anchors had failed during the 22 February 2011 earthquake and that this method of anchoring was equally prevalent in both USA and New Zealand. Preliminary findings from this study are reported, with full information to be provided later once the collected information has been properly processed.

## 7.1 Masonry Material Properties

Following a brief discussion on the quality of the material property information used in past URM earthquake strengthening designs in Christchurch, this section briefly summarises the material properties for 293 mortar samples collected from 61 URM building sites and 67 clay bricks collected from 23 URM building sites. Note that building sites were located both within and outside the Christchurch CBD zone.

### 7.1.1 Material properties used in prior earthquake strengthening design

During inspection of the CCC property records that were reviewed as part of the building survey exercise reported herein, little evidence was found of comprehensive site investigations to determine appropriate masonry material proprieties to be used in the design of earthquake strengthening. Instead, it appeared that most conclusions regarding building condition and material properties were based on visual observations. In contrast, extensive material investigation on buildings is routinely conducted in California in order to establish both the existing condition of the buildings and accurate and reliable material properties for used in structural designs.

From a closer review of structural drawings it was identified that in some cases structural engineering drawings did not correspond to the actual building structure. In one case it was evident that the structural drawings failed to capture the actual wall thickness and wall cavity locations, consequently overlooking a number of issues that resulted in the collapse of building elements. These findings underscore the need for a detailed and comprehensive site investigation to be undertaken for building structures and their constituent materials.

### 7.1.2 Mortar compressive strength

The standard method for evaluating mortar compressive strength is detailed in ASTM C 109-08 (2008). This method involves testing of a 50 mm × 50 mm × 50 mm cube mortar sample, which generally cannot be attained in existing buildings as most mortar joints are only 12 to 18 mm thick. Consequently, irregular mortar and plaster samples extracted from Christchurch URM buildings were capped using gypsum plaster and were tested in compression following the procedure reported by Valek and Veiga (2005). The measured mortar compressive strength was then normalised following the procedure detailed in Lumantarna (2011). The mortar compression test setup is shown in Figure 7.1.

**Figure 7.1 Mortar samples and compression test setup**

The average normalised compressive strength of the 293 mortar samples collected from 61 URM building sites in Christchurch was found to be 2.6 MPa, with a strength range from 0.45 MPa to 25.3 MPa. It is expected that the highest readings were for samples containing modern cement mortar used in repointing (remediation) of existing mortar joints, rather than being associated with historic or original mortar.

Based on visual observations and physical properties it was established that the majority of collected mortar samples were lime-based mortar that were mildly leached and were able to be raked out of joints, but that stayed bound. Thus 45 (74%) buildings from which mortar samples were collected were classified according to NZSEE (2006) as 'Soft', having an average compression strength of 1.2 MPa and closely matching the recommended value from NZSEE (2006). Mortar samples from the remaining 16 buildings were similarly classified according to their appearance and physical properties, and had the average compression strength values shown in Table 7.1. It can be seen that NZSEE (2006) slightly over-predicted the compression strength for the 'Firm' samples, and under-predicted the compression strength of the 'Stiff' samples, but that in general there was good correlation between the predicted compressive strengths and the measured values.

**Table 7.1 Mortar compression strength correlated with NZSEE (2006) classification**

| NZSEE (2006) Classification | NZSEE (2006) prescribed range of compressive strength (MPa) | No. of buildings | Average measured compressive strength (MPa) |
|---|---|---|---|
| Stiff | 8.0 | 5 (8%) | 13.7 |
| Firm | 4.0 | 11 (18%) | 3.1 |
| Soft | 1.0 | 45 (74%) | 1.2 |
| Non-cohesive | 0.0 | 0 (0) | 0.0 |
| **Total** | | **61 (100%)** | |

No scratch tests, as prescribed in NZSEE (2011), were conducted on the mortar samples. Instead, all mortar samples were tested to find their compressive strength, and the average mortar strength was found for each building. The lower bound compression strength values given in NZSEE (2011) were used to determine which classification each building should be assigned to, with the results reported in Table 7.2.

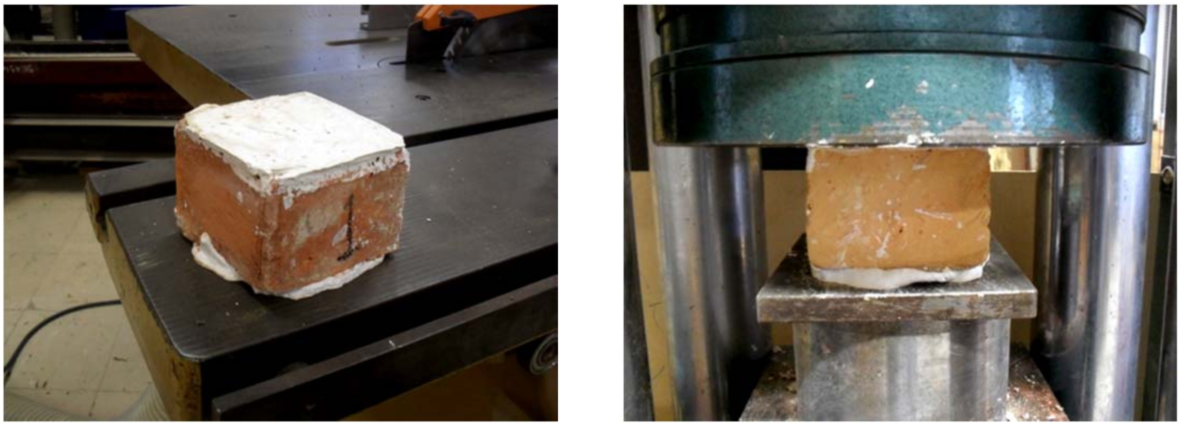
Similarly to the findings using the NZSEE (2006) classifications, Table 7.2 shows that 51 (84%) buildings from which samples were collected and tested had mortar strength corresponding to 'Soft' or to 'Very weak' when using the NZSEE (2011) classifications.

**Table 7.2 Mortar compression strength correlated with NZSEE (2011) classification**

| NZSEE (2011) Classification | Range of compressive strength (MPa) | No. of buildings | Average compressive strength (MPa) |
|---|---|---|---|
| Hard | > 5.8 | 5 (8%) | 13.7 |
| Medium | 3.2 – 5.8 | 5 (8%) | 3.7 |
| Soft | 1.0 - 3.2 | 35 (58%) | 1.7 |
| Very weak | < 1.0 | 16 (26%) | 0.8 |
| **Total** | | **61 (100%)** | |

### 7.1.3 **Brick compressive strength**

The 67 clay bricks extracted from 23 URM building sites in Christchurch were subjected to the laboratory half brick compression test ASTM C 67-03a (2003a). The half brick compression test setup is shown in Figure 7.2.



(a) Half brick sample ready for testing (b) Half brick sample in test machine

**Figure 7.2 Brick sample and compression setup**

All brick units collected from Christchurch buildings were dark red in colour. According to the NZSEE (2006) guideline, bricks showing such colouration have a classification of medium or hard and a corresponding compressive strength of between 20 MPa to 30 MPa, as shown in Table 7.3.

**Table 7.3 Brick compression strength correlated with NZSEE (2006) classification**

| NZSEE (2006) Classification | Range of compressive strength (MPa) | No. of buildings |
|---|---|---|
| Soft | 5 - 10 | 0 (0) |
| Medium | 10 – 20 | 0 (0) |
| Hard | 20 – 30 | 23 (0) |
| **Total** | | **23 (100%)** |

The experimental data for the 67 clay bricks resulted in an average compressive strength of 24.2 MPa, and ranged from 9.5 MPa up to 39.1 MPa. Consequently the NZSEE (2006) criteria adequately predicted the mean of the data set, but inadequately described the strength range.

A similar approach was used for grouping brick samples as was used for the classification of the mortar samples as outlined in section 7.1.2. All bricks from a given building were tested, with the mean value used to classify the brick strength for that building. As for the mortar samples no scratch tests, as prescribed by NZSEE (2011), were conducted on brick samples. Instead the lower bound compression strength values given in NZSEE (2011) were used to determine which classification the bricks from each building should be assigned to, with the results reported in Table 7.4. According to the NZSEE (2011) classification scheme most brick samples were classified as either 'Soft' or 'Medium'.

**Table 7.4 Brick compression strength correlated with NZSEE (2011) classification**

| NZSEE (2011) Classification | Range of compressive strength (MPa) | No. of buildings | Average compressive strength (MPa) |
|---|---|---|---|
| Very soft | 1 – 10 | 1 (4%) | 9.5 |
| Soft | 10 – 20 | 7 (31%) | 17.5 |
| Medium | 20 – 30 | 11 (48%) | 25.6 |
| Hard | > 30 | 4 (17%) | 32.6 |
| Total | | 23 (100%) | |

## 7.2 Studies on the earthquake performance of adhesive anchors in masonry

In this section a brief review of the observed poor performance of adhesive anchors in the 22 February 2011 earthquake is presented, followed by summary details of an experimental program that sought to acquire test data on adhesive anchor pull-out strength

### 7.2.1 Observed earthquake performance of adhesive anchors

Following the 22 February 2011 Christchurch earthquake the poor installation quality of adhesive anchors was identified in at least 10 earthquake strengthened URM buildings located in the Christchurch CBD. In some cases, the reasons for the adhesive anchor failures were apparent, as shown in Figure 7.3(a). The top anchor shown in Figure 7.3(b) is an example of anchor pullout due to insufficient embedment length, while the remaining anchors shown in Figure 7.3(b) indicate a lack of bonding between the anchor and the base material.

(a) Poor installation of adhesive anchors

(b) Recovered adhesive anchors that performed inadequately

**Figure 7.3 Adhesive anchors installation quality**

The observed poor performance of these adhesive anchors (see example in Figure B.1(b)) led to initiation of the test program detailed below.

### 7.2.2 In-field testing of adhesive anchor connections in existing clay brick masonry walls

A collaborative international study was established between researchers at the University of Auckland (NZ) and the University of Minnesota (USA), partly funded by a NSF-RAPID grant, and a research team was deployed to Christchurch during the months of July and August 2011 to conduct in-field tests in order to obtain accurate data on the pullout strength of adhesive type anchors in existing clay brick masonry walls.

Given the difficulties associated with testing existing anchors, the research team opted to test new anchors installed in the exterior façade of exterior walls in existing brick buildings in Christchurch. To test existing anchors would have required the research team to work inside damaged buildings that could potentially sustain additional damage in subsequent aftershocks, to disconnect the existing anchors from roof or floor diaphragms to enable loading of the anchors using the testing equipment, and to provide temporary support to the disconnected wall and diaphragm during the test. Specific objectives of the field test program included identification of the failure modes of adhesive anchors in existing masonry and determination of the influence of the following variables on anchor load-displacement response: type of adhesive, the strength of the masonry materials (brick and mortar), anchor embedment depth, anchor diameter, and use of metal foil sleeve. In addition, the comparative performance of bent anchors (installed at an angle of minimum 22.5$^{o}$ to the perpendicular projection from the wall surface) and anchors positioned horizontally was investigated, as well as the performance of through-bolt anchors with end plate connections. Table 7.5 lists the range of values for the selected variables.

**Table 7.5** Range of values for test parameters in adhesive anchor tests

| Parameter | Range of Values |
|---|---|
| Adhesive type | 3 epoxies and 1 cementitious grout |
| Masonry material strength | Very weak to intermediate strength |
| Anchor embedment depth | 100, 200, 300, 400 (mm) |
| Anchor diameter | 12, 16 (mm) |
| Metal foil sleeve | Yes, No |
| Orientation of anchor | Horizontal and 22.5$^{o}$ to perpendicular projection from wall |

The field test program was conducted using three buildings located in the Wards Brewery Historic Area, which is nestled between Fitzgerald Avenue, Kilmore Street and Chester Street East. The buildings include the original malt house (c. 1881), a malt lot storage building (c. 1910), and one of the barrel storage buildings (c. 1920). All three buildings suffered significant damage during the 2010/2011 earthquakes, and at the time of the field test program they were scheduled for demolition. An indication of the relative strength of the masonry was established based on the building age, visual condition, perceived resistance to drilling and saw cutting, as well as results from in-situ bed joint shear tests. Fifteen bed joint shear tests were conducted, and brick units and mortar samples were extracted and sent to the University of Auckland laboratory for testing.

A total of 170 anchors were installed and tested with the test set-up and loading procedure used to satisfy the New Zealand (SNZ, 2002) and US (ASTM, 2003b) standards, with a typical test arrangement illustrated in Figure 7.4. The tests were conducted using a steel load frame, a manual pump, a loading jack, a load cell, and two displacement transducers (see Figure 7.5). This test procedure enabled the effectiveness of various adhesive anchors to be evaluated. The anchors were mostly DIN 975 class 4.8 steel, with a few anchors cut from DIN 975 grade 8.8 (high-strength) steel. For each combination of test parameters, 5 anchors were installed and tested. Applied tensile force and the corresponding displacement/slip were recorded using a digital data acquisition system. Peak pressure was also recorded manually, and photographs (before and after testing) were taken of all anchors.

**Figure 7.4 Typical test specimen arrangement**

**Figure 7.5 Typical test set-up used for pullout anchor testing**

Some preliminary observations of the field test program are:

- Failure modes included pullout of the anchors (especially in weaker masonry and shorter embedment depths), masonry breakout/anchor pullout (where the leading brick, or part of it, is pulled out with the anchor as shown in Figure 7.6), or anchor yielding (and fracture in some cases);
- Failures approximating the ideal breakout failure, in which rupture occurs in a roughly conical masonry failure surface, were not observed in any of the tests;
- The quality/strength of the masonry was found to be an important variable, as well as the strength of the adhesive, the size of the anchor, and the embedment depth;
- The peak loads recorded during the tests were at least 17.8 kN for the chemical adhesives, and in some cases reached 75.8 kN.


**Figure 7.6** Typical masonry pullout type failure observed

# Section 8:

# Recommendations and closing remarks

## 8.1 Recommendations

1. As reported in Section 5, the performance of parapet restraints was highly variable. In general, restrained parapets performed better than unrestrained parapets. However, there was still a surprisingly high level of damage to restrained parapets. The reasons for this undesirable behaviour should be further investigated.

2. Earthquake strengthening to levels higher than the minimum of 33%NBS have been shown to perform well and to minimise risk to both building occupants and passers-by as well as reduce building damage levels. Based on the data presented in Sections 5 and 6, it is recommended that seismic strengthening of buildings should aim for 100% of the requirement for new buildings, but as a minimum 67%NBS might be acceptable.

3. As shown by the survey results (Sections 5 & 6), buildings that were earthquake strengthened to 33%NBS sustained widely varying levels of damage, but mainly heavy levels of damage. Buildings that were earthquake strengthened to 67%NBS sustained moderate earthquake damage while buildings that were earthquake strengthened to 100%NBS and above typically suffered insignificant earthquake damage. Hence owners that plan to earthquake strengthen their URM buildings to levels below 100%NBS need to be made aware that the minimum requirements are only intended to reduce the life safety hazard and will not necessarily prevent substantial damage to their buildings.

4. It is essential that measures to eliminate brittle failure mechanisms in URM buildings are implemented. These brittle failure mechanisms include out-of-plane and in-plane wall failures, and can cause the catastrophic collapse of a building.

5. Although they are often owned and occupied by different people and companies, where possible URM row buildings should be considered as a whole when undertaking earthquake strengthening.

6. Thorough investigations into building material properties and construction type (for example the determination of whether a building is made of cavity wall construction) need to be conducted as part of any seismic strengthening project. Vulnerable areas such as pounding and potential separation in top corners also need to be addressed.

## 8.2 Closing Remarks

1. Observations reported in the authors' initial report (Report No. 1) indicate that the characteristics of the Christchurch URM building stock located outside the CBD zonation differ somewhat from that reported here, with a greater proportion of URM buildings located outside the CBD having cavity wall construction. The data from these additional URM buildings will need to be amalgamated with the data reported herein in order to obtain a complete understanding of the overall damage to Christchurch URM buildings.

2. Earthquake strengthened buildings generally sustained less damage than buildings that had not been seismically upgraded or had been partially seismically upgraded:

   - The majority of URM buildings that had been earthquake strengthened to a level great than 100%NBS sustained insignificant levels of overall building damage

   - The majority of URM buildings that had been earthquake strengthened to a level between 67% and 100%NBS sustained moderate levels of overall building damage

   - The majority of URM buildings that had been earthquake strengthened to a level between 33% and 67%NBS sustained heavy levels of overall building damage

   - URM buildings that had been earthquake strengthened to a level below 33%NBS sustained variable levels of overall building damage, with the majority of buildings having damage in the heavy or destroyed category.

3. The standard earthquake strengthening procedures for URM buildings do not prevent earthquake damage to these buildings. At best these procedures prevent building collapse.

4. Based on observations, many earthquake strengthened URM buildings still require extensive repair, or even demolition.

5. Type B earthquake strengthening techniques such as shotcrete strengthened walls, additional cross-walls and reinforced concrete or steel strong-backs all performed reasonably well.

# Section 9:

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### 9.2.1 Codes, Model Codes, and Related Documents

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Hoover, Cynthia, 1992, Seismic Retrofit Policies: An Evaluation of Local Practices in Zone 4 and their Application to Zone 3.

This describes interviews with San Francisco Bay Area building officials with a focus on URM bearing wall mitigation programs.

### 9.2.3 Seismic Evaluation and Rehabilitation Guidelines

ASCE, 2003, Seismic Evaluation of Existing Buildings, ASCE/SEI 31-03, Structural Engineering Institute of the American Society of Civil Engineers, Reston, Virginia.

Provides a three-tiered set of analysis procedures for evaluating different structural systems, including URM bearing wall and infill frame buildings.

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Provides detailed evaluation and strengthening performance-based guidelines for existing buildings, including URM bearing wall and infill frame buildings.

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> This provides a comprehensive guidance on the techniques commonly used in seismic rehabilitation, including details and discussion of issues. It is written for engineers with limited experience in seismic rehabilitation or other members of the design community such as architects and project managers coordinating rehabilitation projects or programs to better appreciate the potential scope and construction details of such work.

### 9.2.4 Evaluating the Capacity of Damaged Buildings

FEMA, 1999a, Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings: Basic Procedures Manual, FEMA 306 Report, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C., May.

> This provides detailed evaluation procedures for quantifying the loss of capacity caused by earthquake damaged concrete, reinforced masonry, URM bearing wall, and URM infill frame buildings.

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### 9.2.5 **Earthquake Damage Studies**

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This is a summary of Rutherford & Chekene (1993).

Rutherford & Chekene, 1993, Analysis of Unreinforced Masonry Building Damage Patterns in the Loma Prieta Earthquake and Improvement of Loss Estimation Methodologies: Technical Report to the USGS, March.

Expands the Rutherford & Chekene (1991) study to included extensive correlations with various ground motion parameters and suggestions for improving loss estimation techniques. Funded by the United States Geological Survey.

Rutherford & Chekene, 1997, Development of Procedures to Enhance the Performance of Rehabilitated URM Buildings, prepared by Rutherford & Chekene Consulting Engineers, published by the National Institute of Standards and Technology as Reports NIST GCR 97-724-1 and NIST 97-724-2.

This provides information on damage to retrofitted and unretrofitted URM bearing wall buildings in the 1994 Northridge Earthquake.

### 9.2.6 **Cost and Lost Estimation Studies**

FEMA, 1994, Typical Costs for Seismic Rehabilitation of Existing Buildings, Volume 1: Summary, Second Edition, FEMA 156, Federal Emergency Management Agency, December.

This, together with FEMA 157, summarizes a collection of seismic rehabilitation project costs and provides guidance on how to adjust the costs for specific projects based on different variables. It covers a variety of structural systems.

FEMA, 1995, Typical Costs for Seismic Rehabilitation of Existing Buildings, Volume 2: Supporting Documentation, Second Edition, FEMA 157, Federal Emergency Management Agency, May.

Recht Hausrath, 1990, Seismic Retrofitting Alternatives for San Francisco's Unreinforced Masonry Buildings: Socioeconomic and Land Use Implications of Alternative Requirements, prepared for the San Francisco Department of City Planning.

This was a companion study to Rutherford & Chekene (1990) that provided detailed economic assessments of the viability of the three alternatives that were being considered by the city for a mandatory URM bearing wall strengthening ordinance.

Recht Hausrath, 1993, Socioeconomic and Engineering Study of Seismic Retrofitting Alternatives for Oakland's Unreinforced Masonry Buildings, prepared for the Office of Public Works, City of Oakland.

This is similar to the work done for San Francisco in Rutherford & Chekene (1990) and Recht Hausrath (1990); however, Oakland also included URM infill frame buildings.

Rutherford & Chekene, 1990, Seismic Retrofitting Alternatives for San Francisco's Unreinforced Masonry Buildings: Estimates of Construction Cost & Seismic Damage, for the Department of City Planning of the City and County of San Francisco, Oakland, CA, May.

This was a major engineering study prepared as San Francisco was considering options for a mandatory seismic strengthening ordinance for URM bearing wall buildings. Fifteen prototype buildings were developed to represent the over 2000 URM buildings in the city. Three levels of seismic strengthening were described and retrofit designs were created for each level for each prototype. Cost estimates and loss estimates were performed for all prototypes and levels. The Loma Prieta Earthquake struck as the study was nearing completion and the loss estimation methodology was tested against actual observations in the earthquake.

# Appendix A: [9]

## Terms of Reference – Royal Commission of Inquiry into Building Failure Caused by the Canterbury Earthquake

Elizabeth the Second, by the Grace of God Queen of New Zealand and her Other Realms and Territories, Head of the Commonwealth, Defender of the Faith:

To The Honourable MARK LESLIE SMITH COOPER, of Auckland, Judge of the High Court of New Zealand; Sir RONALD POWELL CARTER, KNZM, of Auckland, Engineer and Strategic Adviser; and RICHARD COLLINGWOOD FENWICK, of Christchurch, Associate Professor of Civil Engineering:

GREETING:

*Recitals*

WHEREAS the Canterbury region, including Christchurch City, suffered an earthquake on 4 September 2010 and numerous aftershocks, for example—

(a) the 26 December 2010 (or Boxing Day) aftershock; and

(b) the 22 February 2011 aftershock:

WHEREAS approximately 180 people died of injuries suffered in the 22 February 2011 aftershock, with most of those deaths caused by injuries suffered wholly or partly because of the failure of certain buildings in the Christchurch City central business district (**CBD**), namely the following 2 buildings:

(a) the Canterbury Television (or CTV) Building; and

(b) the Pyne Gould Corporation (or PGC) Building:

WHEREAS other buildings in the Christchurch City CBD, or in suburban commercial or residential areas in the Canterbury region, failed in the Canterbury earthquakes, causing injury and death:

WHEREAS a number of buildings in the Christchurch City CBD have been identified as unsafe to enter following the 22 February 2011 aftershock, and accordingly have been identified with a red card to prevent persons from entering them:

[9] Downloaded from: http://canterbury.royalcommission.govt.nz/vwluResources/PCO%2015148v2%20-%20Terms%20of%20Reference%20(doc)/$file/PCO%2015148v2%20-%20Terms%20of%20Reference.doc

WHEREAS the Department of Building and Housing has begun to investigate the causes of the failure of 4 buildings in the Christchurch City CBD (the **4 specified buildings**), namely the 2 buildings specified above, and the following 2 other buildings:

(a) the Forsyth Barr Building; and

(b) the Hotel Grand Chancellor Building:

WHEREAS it is desirable to inquire into the building failures in the Christchurch City CBD, to establish—

(a) why the 4 specified buildings failed severely; and

(b) why the failure of those buildings caused such extensive injury and death; and

(c) why certain buildings failed severely while others failed less severely or there was no readily perceptible failure:

WHEREAS the results of the inquiry should be available to inform decision-making on rebuilding and repair work in the Christchurch City CBD and other areas of the Canterbury region:

*Appointment and order of reference*

KNOW YE that We, reposing trust and confidence in your integrity, knowledge, and ability, do, by this Our Commission, nominate, constitute, and appoint you, The Honourable MARK LESLIE SMITH COOPER, Sir RONALD POWELL CARTER, and RICHARD COLLINGWOOD FENWICK, to be a Commission to inquire into and report (making any interim or final recommendations that you think fit) upon (having regard, in the case of paragraphs (a) to (c), to the nature and severity of the Canterbury earthquakes)—

*Inquiry into sample of buildings and 4 specified buildings*

(a) in relation to a reasonably representative sample of buildings in the Christchurch City CBD, including the 4 specified buildings as well as buildings that did not fail or did not fail severely in the Canterbury earthquakes—

(i) why some buildings failed severely; and

(ii) why the failure of some buildings caused extensive injury and death; and

(iii) why buildings differed in the extent to which—

(A) they failed as a result of the Canterbury earthquakes; and

(B) their failure caused injury and death; and

(iv) the nature of the land associated with the buildings inquired into under this paragraph and how it was affected by the Canterbury earthquakes; and

(v) whether there were particular features of a building (or a pattern of features) that contributed to whether a building failed, including (but not limited to) factors such as—

(A) the age of the building; and

(B) the location of the building; and

(C) the design, construction, and maintenance of the building; and

(D) the design and availability of safety features such as escape routes; and

(b) in relation to all of the buildings inquired into under paragraph (a), or a selection of them that you consider appropriate but including the 4 specified buildings,—

(i) whether those buildings (as originally designed and constructed and, if applicable, as altered and maintained) complied with earthquake-risk and other legal and best-practice requirements (if any) that were current—

(A) when those buildings were designed and constructed; and

(B) on or before 4 September 2010; and

(ii) whether, on or before 4 September 2010, those buildings had been identified as “earthquake-prone” or were the subject of required or voluntary measures (for example, alterations or strengthening) to make the buildings less susceptible to earthquake risk, and the compliance or standards they had achieved; and

(c) in relation to the buildings inquired into under paragraph (b), the nature and effectiveness of any assessment of them, and of any remedial work carried out on them, after the 4 September 2010 earthquake, or after the 26 December 2010 (or Boxing Day) aftershock, but before the 22 February 2011 aftershock; and

*Inquiry into legal and best-practice requirements*

(d) the adequacy of the current legal and best-practice requirements for the design, construction, and maintenance of buildings in central business districts in New Zealand to address the known risk of earthquakes and, in particular—

(i) the extent to which the knowledge and measurement of seismic events have been used in setting legal and best-practice requirements for earthquake-risk management in respect of building design, construction,
and maintenance; and

(ii) the legal requirements for buildings that are "earthquake-prone" under section 122 of the Building Act 2004 and associated regulations, including—
  (A) the buildings that are, and those that should be, treated by the law as "earthquake-prone"; and
  (B) the extent to which existing buildings are, and should be, required by law to meet requirements for the design, construction, and maintenance of new buildings; and
  (C) the enforcement of legal requirements; and

(iii) the requirements for existing buildings that are not, as a matter of law, "earthquake-prone", and do not meet current legal and best-practice requirements for the design, construction, and maintenance of new buildings, including whether, to what extent, and over what period they should be required to meet those requirements; and

(iv) the roles of central government, local government, the building and construction industry, and other elements
of the private sector in developing and enforcing legal and best-practice requirements; and

(v) the legal and best-practice requirements for the assessment of, and for remedial work carried out on, buildings after any earthquake, having regard to lessons from the Canterbury earthquakes; and

(vi) how the matters specified in subparagraphs (i) to (v) compare with any similar matters in other countries; and

*Other incidental matters arising*

(e) any other matters arising out of, or relating to, the foregoing that come to the Commission's notice in the course of its inquiries and that it considers it should investigate:

*Matters upon or for which recommendations required*

And, without limiting the order of reference set out above, We declare and direct that this Our Commission also requires you to make both interim and final recommendations upon or for—

(a) any measures necessary or desirable to prevent or minimise the failure of buildings in New Zealand due to earthquakes likely to occur during the lifetime of those buildings; and

(b) the cost of those measures; and

(c) the adequacy of legal and best-practice requirements for building design, construction, and maintenance insofar as those requirements apply to managing risks of building failure caused by earthquakes:

*Exclusions from inquiry and scope of recommendations*

But, We declare that you are not, under this Our Commission, to inquire into, determine, or report in an interim or final way upon the following matters (but paragraph (b) does not limit the generality of your order of reference, or of your required recommendations):

(a) whether any questions of liability arise; and

(b) matters for which the Minister for Canterbury Earthquake Recovery, the Canterbury Earthquake Recovery Authority, or both are responsible, such as design, planning, or options for rebuilding in the Christchurch City CBD; and

(c) the role and response of any person acting under the Civil Defence Emergency Management Act 2002, or providing any emergency or recovery services or other response, after the 22 February 2011 aftershock:

*Definitions*

And, We declare that, in this Our Commission, unless the context otherwise requires,—

**best-practice requirements** includes any New Zealand, overseas country's, or international standards that are not legal requirements

**Canterbury earthquakes** means any earthquakes or aftershocks in the Canterbury region—

(a) on or after 4 September 2010; and

(b) before or on 22 February 2011

**Christchurch City CBD** means the area bounded by the following:

(a) the 4 avenues (Bealey Avenue, Fitzgerald Avenue, Moorhouse Avenue, and Deans Avenue); and

(b) Harper Avenue

**failure**, in relation to a building, includes the following, regardless of their nature or level of severity:

(a) the collapse of the building; and

(b) damage to the building; and

(c) other failure of the building

**legal requirements** includes requirements of an enactment (for example, the building code):

*Appointment of chairperson*

And We appoint you, The Honourable MARK LESLIE SMITH COOPER, to be the chairperson of the Commission:

*Power to adjourn*

And for better enabling you to carry this Our Commission into effect, you are authorised and empowered, subject to the provisions of this Our Commission, to make and conduct any inquiry or investigation under this Our Commission in the manner and at any time and place that you think expedient, with power to adjourn from time to time and from place to place as you think fit, and so that this Our Commission will continue in force and that inquiry may at any time and place be resumed although not regularly adjourned from time to time or from place to place:

*Information and views, relevant expertise, and research*

And you are directed, in carrying this Our Commission into effect, to consider whether to do, and to do if you think fit, the following:

(a) adopt procedures that facilitate the provision of information or views related to any of the matters referred to in the order of reference above; and

(b) use relevant expertise, including consultancy services and secretarial services; and

(c) conduct, where appropriate, your own research; and

(d) determine the sequence of your inquiry, having regard to the availability of the outcome of the investigation by the Department of Building and Housing and other essential information, and the need to produce an interim report:

*General provisions*

And, without limiting any of your other powers to hear proceedings in private or to exclude any person from any of your proceedings, you are empowered to exclude any person from any hearing, including a hearing at which evidence is being taken, if you think it proper to do so:

And you are strictly charged and directed that you may not at any time publish or otherwise disclose, except to His Excellency the Governor-General of New Zealand in pursuance of this Our Commission or by His Excellency's direction, the contents or purport of any interim or final report so made or to be made by you:

And it is declared that the powers conferred by this Our Commission are exercisable despite the absence at any time of any 1 member appointed by this Our Commission, so long as the Chairperson, or a member deputed by the Chairperson to act in the place of the Chairperson, and at least 1 other member, are present and concur in the exercise of the powers:

*Interim and final reporting dates*

And, using all due diligence, you are required to report to His Excellency the Governor-General of New Zealand in writing under your hands as follows:

(a) not later than 11 October 2011, an interim report, with interim recommendations that inform early decision-making on rebuilding and repair work that forms part of the recovery from the Canterbury earthquakes; and

(b) not later than 11 April 2012, a final report:

And, lastly, it is declared that these presents are issued under the authority of the Letters Patent of Her Majesty Queen Elizabeth the Second constituting the office of Governor-General of New Zealand, dated 28 October 1983*, and under the authority of and subject to the provisions of the Commissions of Inquiry Act 1908, and with the advice and consent of the Executive Council of New Zealand.

In witness whereof We have caused this Our Commission to be issued and the Seal of New Zealand to be hereunto affixed at Wellington this 11th day of April 2011.

Witness Our Trusty and Well-beloved The Right Honourable Sir Anand Satyanand, Chancellor and Principal Knight Grand Companion of Our New Zealand Order of Merit, Principal Companion of Our Service Order, Governor-General and Commander-in-Chief in and over Our Realm of New Zealand.

ANAND SATYANAND, Governor-General.

By His Excellency's Command—

JOHN KEY, Prime Minister.

Approved in Council—

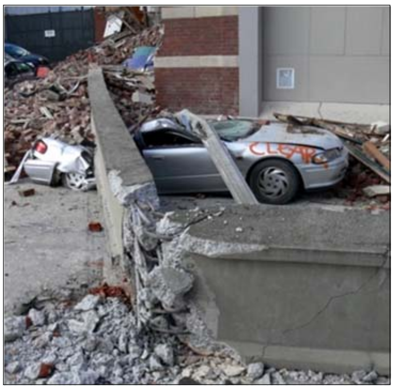
REBECCA KITTERIDGE, Clerk of the Executive Council.

# Appendix B:

# Observed performance of parapet restraints

Although the different types of restraints used to secure the parapets of URM buildings in the Christchurch CBD were difficult to identify during building surveying, such that this parameter was excluded from the building database reported herein, a number of concrete capping beams were observed to have fully collapsed and caused significant damage at ground level. These falling concrete capping beams also initiated collapse of other parts of the building because of their length and because they were often connected to other structural elements (see Figure B.1(a) for an example, and Figure 2.3 for drawing schematics from FEMA 547).

Flexible parapet restraint rods that were connected to the parapet using a single adhesive anchor typically performed poorly, failing to provide sufficient restraint to the parapet (see Figure B.1(b) and also section 7.2). However, parapet restraint using rigid steel braces connected to a continuous steel section and braced back to the roof structure performed well and in most observed cases prevented collapse of the parapets (see Figure B.1(c-d)). Best performance was observed for parapet restraints which had steel angels 'confining' the parapet and bracing it back to the roof via rigid braces, as shown in Figure B.1(e).



(a) Collapse of concrete capping beam used to replace the original masonry parapet

(b) Thin and flexible restraint rods failed to restrain the parapet

(c) Well restrained parapet

(d) Parapet restraint using rigid steel braces connected to a continuous steel section

(e) Steel angels ‘confining’ the parapet and securing it back to the roof via rigid braces

**Figure B.1 Performance of parapet restraint types**