Report to the Royal Commission of Inquiry

The Performance of Unreinforced Masonry Buildings in the 2010/2011 Canterbury Earthquake Swarm

by

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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
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)
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
Earthquake Prone Building	A building having an expected earthquake performance that is less than 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)
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
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
Magnitude	A measure of the total energy released by the earthquake, originally based upon the Richter Scale but now determined using a revised technique
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 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)
Period	A property that describes how the building will shake 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
Seismic zone factor	A factor that numerically describes the seismicity of a region
Solid construction	Wall construction where multiple leafs (or layers) of masonry are used to create the wall thickness, without including a cavity
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
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
%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

The scope and purpose of this report were established at a meeting on 19 July 2011 with the members of the Royal Commission of Inquiry into Building Failure Caused by the Canterbury Earthquakes. The purpose of this report is to provide a resource, both for the members of the Royal Commission of Inquiry and for other parties wishing to make a submission to the Commission when hearings begin. It was established that the scope would include:

- Details of the characteristics and value of the New Zealand unreinforced masonry (URM) building stock and of the assessed seismic vulnerability of this building stock;
- Details of the performance of URM buildings within the Christchurch Central Business District (CBD) in the 2010/2011 Canterbury earthquake swarm;
- Information on technologies (including costs) available for the seismic improvement of URM buildings, and on the hierarchy of improvements that may be applied in order to improve the seismic performance of URM buildings;
- Identify URM buildings that are or were representative of their class of building and whose observed earthquake performance was representative of how that class of building would behave during earthquake actions throughout the rest of New Zealand;
- Comments on the adequacy of current practices and methodologies that may be adopted in response to the events in Christchurch.

In an effort to provide the information required by the Royal Commission, the authors have drawn on information obtained during their work with building damage assessment teams following the 4 September 2010 and 22 February 2011 earthquakes as well as data and information collected from reference material that is acknowledged in the report. Two items of interest to the Commission:

a) URM building damage statistics from the 22 February 2011 earthquake; and

b) costings for various seismic retrofit technologies that have been shown to be effective

are still being compiled and are not provided in this preliminary report. It is expected that this information will be available in time for inclusion in the final report.

In brief, the main recommendations of this report are:

- All URM buildings should be improved so that ~~the public is protected from all~~ falling hazards ~~such as~~ chimneys, parapets, gable end walls and out-of-plane wall failures. These parts of URM buildings should be improved to the full design strength required for new buildings in New Zealand. If required, further building improvements should aim for 100% of the requirements for new buildings with lower values negotiable on a case by case basis. However, a minimum of 67% is recommended.
- There should be a single, national policy for URM building maintenance and seismic strengthening rather than multiple regional policies.
- The estimated cost to upgrade all of New Zealand's approximately 3867 URM buildings to a minimum of 67% of the NBS requirements is approximately $2 billion. This is slightly more than the estimated value of $1.5 billion for the total URM building stock. Clearly, a cost effective strategy is needed to direct the limited resources available to tackle this problem.
- Field testing of a limited number of existing URM buildings in the Christchurch CBD or nearby (that have been listed for demolition) would improve the current understanding of the seismic capacity of these buildings as well as offer an opportunity to develop and validate more cost-effective seismic strengthening/retrofit technologies. Such testing would focus on global structural performance characteristics and how loads are transmitted through buildings, and would be undertaken using such techniques as snap back testing to generate lateral loads and deformations that simulate earthquake effects. The performance of structural elements either extracted from such buildings, or tested in place, would also provide important new information.
- In view of the estimated cost to upgrade all URM buildings to a minimum of 67% of the NBS, it is proposed that first priority be given to ensuring public safety by securing/removing falling hazards as outlined in section 7: Recommendation 3, Stage 1 and Stage 2. The cost to do this is unknown but would be substantially less than the amount to fully upgrade all buildings.

Section 1:

Introduction and background

This section provides introductory information on the scope and purpose of this report, followed by details of the early European settlement of Christchurch. The details provided on masonry construction practices in early Christchurch are a prelude to the critique of the architectural characteristics and the number and seismic vulnerability of the New Zealand unreinforced masonry (URM) building stock that is reported in section 2. Background information on the evolution of New Zealand building codes, with particular attention given to provisions for seismic improvement of existing buildings, is next provided. The section concludes with some brief comments on the seismological characteristics of the 2010/2011 Canterbury earthquake swarm, and in particular information is provided to explain that most URM buildings in Christchurch were subjected to earthquake loads that were well in excess of the assessed earthquake strength of the Christchurch URM building stock.

1.1 Scope and Purpose

The scope and purpose of this report were established at a meeting held on 19 July 2011 with the members of the Royal Commission of Inquiry into Building Failure Caused by the Canterbury Earthquakes. The purpose of this report is to provide a resource, both for the members of the Royal Commission of Inquiry and for other parties wishing to make a submission to the Commission when hearings begin. The scope includes but is not necessarily limited to:

- Details of both stone masonry and clay brick URM buildings, including both iconic buildings and more regular buildings;

- Details of the architectural and structural characteristics of the URM building stock of New Zealand, with particular emphasis on the uniform characteristics of these buildings throughout New Zealand and on their role in defining village atmosphere as local centres in larger cities and as the principal commercial location of smaller cities and towns throughout New Zealand;
- Details of the value of the New Zealand URM building stock and of the assessed seismic vulnerability of this building stock;
- Details of the performance of URM buildings in the 2010/2011 Canterbury earthquake swarm, with particular but not exclusive attention given to the performance of the buildings located within the Christchurch Central Business District (CBD) as defined within the Terms of Reference of the Inquiry as the area bounded by Bealey Avenue, Fitzgerald Avenue, Moorhouse Avenue, Deans Avenue and Harper Avenue. These details include representative examples of failure modes that were observed;
- Statistics on the observed earthquake performance of the URM building stock, and a report of data on post-earthquake building demolitions, primarily pertaining to URM buildings;
- Identification of URM buildings that are or were both representative of their class of building and whose observed earthquake performance was representative. This selection of representative buildings is to include both unretrofitted and retrofitted stone and clay brick URM buildings, and both buildings that performed poorly and buildings that performed well;
- Information on technologies available for the seismic improvement of URM buildings, and on the hierarchy of improvements that may be applied in order to improve the seismic performance of URM buildings;
- Where available, information on the cost of implementing improvements to the national URM building~~s~~ stock;
- Comments on the adequacy or inadequacy of current practices and on methodologies that may be adopted in response to the events in Christchurch.

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

1.2 European settlement of Christchurch

1.2.1 Early Christchurch construction

Construction in the early period of colonisation was primarily of timber for residential and smaller commercial buildings due to the proximity and abundance of the local resource in the Papanui and Riccarton Forest. In the late 1850s Christchurch prospered from the wool trade and this allowed the transition from wood to stone and clay brick masonry for the construction of public buildings. The spirit in which the Canterbury settlement was founded instructed a building style that imitated the style of the home country (Wilson, 1984). The city's second town hall was built in stone in 1862-1863, the first stone building of Christ's College was constructed in 1863, and the city's architectural jewel, the stone Provincial Council Chambers, was completed in 1864 (Wilson, 1984). The aesthetic quality of Christchurch city was also regulated in terms of building size and style in order to maintain a regular appearance. In the 1860s and right through to the 1880s a vogue for Venetian Gothic architecture for commercial buildings was indulged, distinguishing the buildings of Christchurch from those of other New Zealand cities that were embracing classical and Renaissance styles. The city was populated with mostly two and three storey buildings that were complementary in height to their neighbouring buildings. This regularity in style and size was accentuated by the rigid regular gridded streets. Construction slowed during a period of economic depression in the 1870s, but allowed for a new period of design to develop by the time that prosperity returned in the late 1890s (Rice, 2008).

Figure 1.1 Victorian Christchurch in 1885 (Coxhead, 1885)

By 1914 the central area of Christchurch had been largely rebuilt, resulting in a city that was “interesting for its architectural variety, pleasing for its scale and distinctively New Zealand” (Wilson, 1984). Figure 1.1 and Figure 1.2 show photos of historical Christchurch from 1885 and 1910 respectively. Two of the many influential architects of Christchurch were J. C. Maddison (1850-1923), whose design focus was inspired by the Italianate style, and J. J. Collins (1855–1933), who in partnership with R. D. Harman (1859-1927) chose brick masonry as their medium for large commercial and institutional buildings. By the 1920s wooden structures in the city were rare, and were seen as small irregular relics of the past.

Figure 1.2 New Zealand Express Company building, Christchurch's first 'skyscraper', photo circa 1910 (Brittenden Collection, 1910)

1.2.2 Rise and decline of unreinforced masonry construction

Brick masonry construction was seen as a symbol of permanency, when compared with the construction of timber buildings. The use of masonry was further justified after a number of fires in inner city Christchurch during the 1860s. The centre of Lyttelton was also destroyed in 1870 (Christchurch City Libraries, 2006; Wilson, 1984). The fire-proof nature of masonry led to it being readily adopted as the appropriate building material for high importance structures such as government buildings, schools, churches, and the Press building that housed the local newspaper company.

In Christchurch's founding years, the city and its surrounding boroughs were subjected to three medium sized earthquakes, and as many as seven smaller earthquakes that were centred closer to the north of the South Island (GeoNet, 2010). The earthquake of 5th June 1869 was the most damaging to the settlement of Christchurch, causing damage to chimneys, government buildings, churches and homes throughout the central city and the surrounding boroughs of Avon (Avonside), Linwood, Fendalton and Papanui (Christchurch City Libraries, 2006). The worst of the damage reported was to the stone spire of St John's church in Latimer Square which was cracked up its entire height (Rice, 2008). In Government buildings, the tops of two chimneys came down, plaster was cracked, and several stones were displaced. Similar damage occurred in some other brick and stone masonry buildings, including Matson's building, the NZ Loan & Trust building and the NZ Insurance building. The majority of the damage to houses was the result of brick chimneys toppling and in one case the exterior brick wall of a house in Manchester Street collapsed. The damage was most intense within the inner confines of the city, decreasing from a MM 7[1] intensity in the city to MM 5 at Kaiapoi and Halswell. However, a few chimneys and household contents were also damaged at Lyttelton (Christchurch City Libraries, 2006). Twelve years later another earthquake was felt in Christchurch, but resulted in less damage than the previous 1869 earthquake (GeoNet, 2010). The only reported damage from the 1881 earthquake was that to the spire of the Cathedral, which was still in construction.

The large earthquake that struck the Amuri District of Canterbury (about 100 km north of Christchurch) in 1888 is thought to have originated on the Hope Fault, which is part of the Marlborough Fault Zone (Stirling, 2008). The earthquake's intensity reached MM 9 in the epicentre area, and caused severe damage to buildings made of cob and stone masonry located in the Amuri District (now part of the Hurunui Territoiral Authority of Canterbury), as well as in Hokitika and Greymouth. This earthquake was felt in Christchurch city, and caused minor damage to buildings (PapersPast, 2010). A later earthquake in 1901 centred in Cheviot damaged the spire on the Cathedral for the third time in its short life and led to reconstruction of the spire in timber. Figure 1.3 shows the damage to the spire from the 1888 and the 1901 earthquakes.

Although these earthquakes early in the development of Christchurch did result in some damage to buildings, and in particular to stone and clay brick masonry buildings, none of these earthquakes had an effect on the construction and design of buildings as did the 1931 Hawke's Bay earthquake (see section 2.1.2).

1.3 The evolution of New Zealand building codes

The construction of URM buildings in New Zealand peaked in the decade between 1920 and 1930 and subsequently declined (see Figure B.3 and Figure B.4), with one of the most important factors in this decline being the economic conditions of the time. The Great Depression in the 1930s and the outbreak of World War II significantly slowed progress in the construction sector, and few large buildings of any material were constructed in the period between 1935 and 1955 (Stacpoole & Beaven, 1972; Megget, 2006). Equally important in the history of URM buildings in New Zealand was the 1931 M7.8 Hawke's Bay earthquake, and the changes in building provisions which it precipitated.

[1] The Modified Mercalli intensity scale is a seismic scale used for measuring the intensity of an earthquake. The scale measures the *effects* of an earthquake, and is distinct from the moment magnitude M_w usually reported for an earthquake (from http://en.wikipedia.org/wiki/Mercalli_intensity_scale)

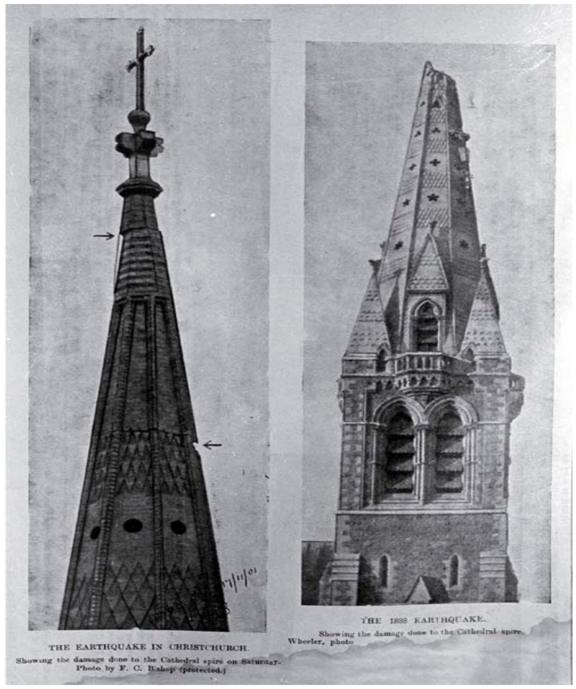
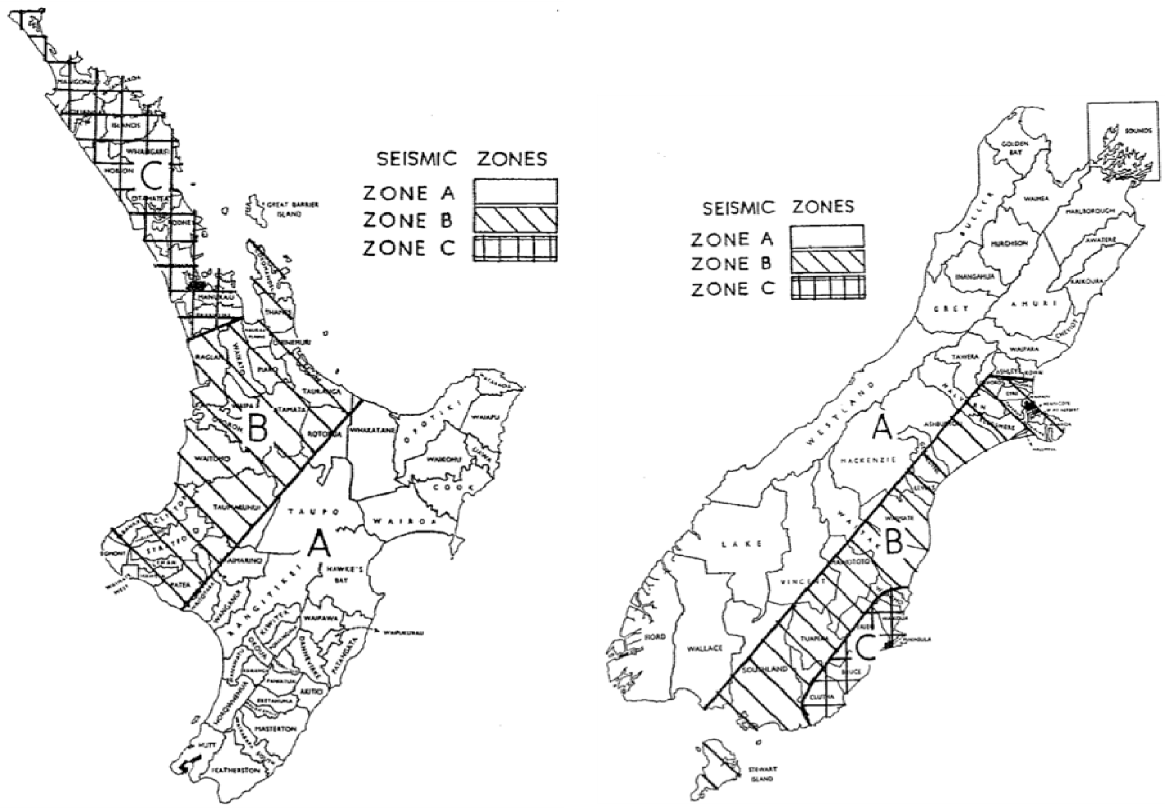


Figure 1.3 Damage to the Cathedral Spire in the 1888 (left) and 1901 (right) earthquakes (Bishop & Wheeler, 1901).

The destruction of many URM buildings in Napier graphically illustrated that URM construction possessed insufficient strength to resist lateral forces induced in an earthquake due to its brittle nature and inability to dissipate energy. Later in 1931, in response to that earthquake, the Building Regulations Committee presented a report to the Parliament of New Zealand entitled "Draft General Building By-Law" (Cull, 1931). This development was the first step towards requiring seismic provisions in the design and construction of new buildings. In 1935, this report evolved into NZSS no. 95, published by the newly formed New Zealand Standards Institute, and required a ~~horizontal acceleration for design of 0.1g~~, and this requirement applied to the whole of New Zealand (New Zealand Standards Institute, 1935). NZSS no. 95 also suggested that buildings for public gatherings should have frames constructed of reinforced concrete or steel. The By-Law was not enforceable, but it is understood that it was widely used especially in the larger centres of Auckland, Napier, Wellington, Christchurch and Dunedin (Megget, 2006).

The provisions of NZSS no. 95 were confined to new buildings only, but the draft report acknowledged that strengthening of existing buildings should also be considered, and that alterations to existing buildings were required to comply (Davenport, 2004). In 1939 and 1955 new editions of this By-Law were published, and apart from suggesting in 1955 that the seismic coefficient vary linearly from zero at the base to 0.12 at the top of the building (formerly the seismic coefficient was uniform up the height of the building), there were few significant changes (Beattie et al., 2008). It was not until 1965 that much of the recent research at the time into seismic design was incorporated into legislation. The New Zealand Standard Model Building By-Law NZSS 1900 Chapter

8:1965 explicitly prohibited the use of URM: (a) in Zone A; (b) of more than one storey or 15 ft (4.6 m) eaves height in Zone B; (c) of more than two storeys or 25 ft (7.6 m) eaves height in Zone C. These zones refer to the seismic zonation at the time, which have subsequently changed and evolved. Zone A consisted of regions of the highest seismic risk and Zone C consisted of regions of the lowest seismic risk (New Zealand Standards Institute, 1965). Details of the seismic zonation in NZSS 1900 are shown in Figure 1.4. Again, the provisions of this By-Law did not apply automatically and had to be adopted by local authorities.



(a) North Island **(b) South Island**

Figure 1.4 Map of seismic zones (from NZSS 1900 Chapter 8:1965)

The 1965 code required that buildings be designed and built with "adequate ductility", although further details were not given. The next version of the loadings code was published in 1976 as NZS 4203 (Standards Association of New Zealand, 1976), and was a major advance on the 1965 code. Most importantly, the 1976 loadings code was used in conjunction with revised material codes: steel, reinforced concrete, timber and reinforced masonry, which all required specific detailing for ductility. Thus after the publication of this code in 1976, unreinforced masonry was explicitly prohibited as a building material throughout the whole of New Zealand.

The use of URM was implicitly discouraged through legislation from as early as 1935, and although it was still allowed in some forms after 1965, observations of existing building stock show its minimal use from 1935 onwards, especially for larger buildings.

This is thought to be significantly attributable to the exceptionally rigorous quality of design and construction by the Ministry of Works at the time (Megget, 2006; Johnson, 1963). Although two storey URM buildings were permitted in Auckland (Zone C) after 1965, only three existing URM buildings in Auckland City constructed after 1940 have been identified. All three are single storey and they were constructed in 1950, 1953 and 1955.

1.3.1 Provisions for the seismic upgrade of existing buildings

As building codes were being developed for the design of new buildings, attention was also given to the performance of existing buildings in earthquakes. The first time this was addressed in legislation was Amendment 301A to the 1968 Municipal Corporations Act (New Zealand Parliament, 1968). This Act allowed territorial authorities, usually being boroughs, cities or district councils, to categorise themselves as earthquake risk areas and thus to apply to the government to take up powers to classify earthquake prone buildings and require owners to reduce or remove the danger. Buildings (or parts thereof) of high earthquake risk were defined as being those of unreinforced concrete or unreinforced masonry with insufficient capacity to resist earthquake forces that were 50% of the magnitude of those forces defined by NZS 1900 Chapter 8:1965. If the building was assessed as being "potentially dangerous in an earthquake", the council could then require the owner of the building within the time specified in the notice to remove the danger, either by securing the building to the satisfaction of the council, or if the council so required, by demolishing the building.

Most major cities and towns took up the NZS 1900 Chapter 8:1965 legislation, and as an indication of the effect of this Act, between 1968 and 2003 Wellington City Council achieved strengthening or demolition of 500 out of 700 buildings identified as earthquake prone (Hopkins et al., 2008). Auckland City Council, in spite of having a low seismicity, took a strong interest in the legislation and this led to considerable activity in strengthening buildings (see Boardman, 1983). In Christchurch, a moderately high seismic zone, the City Council implemented the legislation, but adopted a more passive approach, generally waiting for significant ~~developments~~ to trigger the requirements. In Dunedin, now seen to be of low seismic risk, little was done in response to the 1968 legislation although strengthening of schools, public buildings and some commercial premises was achieved. As a result, Dunedin has a high percentage of URM buildings compared with many other cities in New Zealand (Hopkins, 2009). Megget (2006) and Thornton (2010) state that much of the strengthening in Wellington was accomplished with extra shear walls, diagonal bracing or buttressing and the tying of structural floors and walls together, and that many brittle hazards such as parapets and clock towers had been removed after the two damaging 1942 South Wairarapa earthquakes (M7 & M7.1) which were felt strongly in Wellington. Hopkins et al. (2008) noted that:

> "there was criticism at the loss of many older heritage buildings and at the use of intrusive retrofitting measures which were not harmonious with the architectural fabric of the building (McClean, 2009). At the same time, this did provide an opportunity in many cases for the land

> on which the old building was situated to be better utilised with new, larger and more efficiently designed structures."
>
> "A major drawback of the 1968 legislation, which endured until 2004, surviving intact with the passage of the Building Act 1991, was that the definition of an earthquake prone building and the required level to which such buildings should be improved remained tied to the 1965 code. Most territorial authorities called for strengthening to one-half or two-thirds of the 1965 code, and many buildings which were strengthened to these requirements were subsequently found to fall well short of the requirements of later design standards for new buildings" (Hopkins et al., 2008).

Wellington City Council found that in January 2008, of 97 buildings which had been previously strengthened, 61 (63%) were subsequently identified as potentially earthquake prone (Stevens & Wheeler, 2008; Bothara et al., 2008). This situation was recognised by the New Zealand Society for Earthquake Engineering (NZSEE), who were also concerned about the performance of more modern buildings, particularly after the observed poor performance of similarly aged buildings in earthquakes in Northridge, California (1994) and Kobe, Japan (1995). NZSEE pushed for new, more up-to-date and wide-ranging legislation. This initiative was supported by the Building Industry Authority, later to become part of the Department of Building and Housing, and a new Building Act came into effect in August 2004 (New Zealand Parliament, 2004). This development brought in new changes as to what constituted an "Earthquake Prone Building". In particular, the definition of an earthquake prone building was tied to the current design standard of the time, and no longer to the design standard of any particular year. The legislation allowed any territorial authority to require the owner of an earthquake prone building to take action to reduce or remove the danger. Each territorial authority was required to have a policy on earthquake prone buildings, and to consult publicly on this policy before its adoption. Policies were required to address the approach and priorities and to state what special provisions would be made for heritage buildings. The 2004 legislation applied to all building types except residential ones, (residential buildings were excluded unless they comprised 2 or more storeys and contained 3 or more household units).

As soon as the 1968 legislation to attempt to mitigate the effects of earthquake prone buildings came into effect, the New Zealand National Society for Earthquake Engineering (NZNSEE) set up a steering committee to provide a code of practice in an effort to assist local authorities to implement the legislation. Since the first draft code of practice published by the NZNSEE (1972), several successive publications have been produced, each extending on the previous version. These guidelines have been instrumental in helping engineers and territorial authorities to assess the expected seismic performance of existing buildings consistent with the requirements of the legislation. Guidelines for assessing and upgrading earthquake risk buildings were published as a bulletin article in 1972 (NZNSEE, 1972) and then separately published the following year, which became colloquially known as the "Brown Book" (NZNSEE, 1973). This document provided guidelines for surveying earthquake risk buildings and for the identification of particularly hazardous buildings and features. The document did not establish or recommend strength levels to which earthquake prone buildings should be upgraded, and thus standards varied from one area to another. It was implicit that strengthening be to more than half the standard required in Chapter 8 of the 1965 NZSS Model Building By-Law.

In 1982, NZSEE established a study group to examine and rationalise the use of these guidelines and to produce further guidelines and recommendations. This activity culminated in the publication in 1985 of what became known as the "1985 Red Book" (NZNSEE, 1985). Again, this document was primarily of a technical nature and the responsibilities of what to do with buildings still rested with local authorities. The publication was intended to promote a consistent approach throughout New Zealand for the strengthening of earthquake risk buildings and included a recommended level to which buildings should be strengthened plus the time scale to complete the requirements. The basic objective was to establish a reasonably consistent reduction of the overall risk to life which the country's stock of earthquake risk buildings represented. Based on overseas experiences, particularly in Los Angeles in Southern California, a philosophy was accepted of providing owners of earthquake risk buildings with the option of interim securing to gain limited extension of useful life, after which the building should be strengthened to provide indefinite future life. The design of interim securing systems was to be based on minimum seismic coefficients which represented two-thirds of those specified in NZSS 1900, Chapter 8 (New Zealand Standards Institute, 1965). For "permanent" strengthening measures, it was recommended that the building be strengthened to the standard of a new building, but with the design lateral forces reduced depending on the occupancy classification and type of strengthening system. This publication was widely used by territorial authorities and designers.

In 1992 the NZNSEE again set up a study group to review the 1985 publication, and this resulted in another publication, which similarly became colloquially known as the "1995 Red Book" (NZNSEE, 1995). This document extended the approach and content of its predecessor and took into account the changing circumstances, technical developments and improved knowledge of the behaviour of URM buildings in earthquakes. In particular, earthquake risk buildings in that document were taken to include all unreinforced masonry buildings, and not just those which were defined as "earthquake prone" in terms of the Building Act of the time, which still referred back to the 1965 code. Another key difference from the 1985 Red Book was that a single stage approach to strengthening was suggested, in contrast to the two stage securing and strengthening procedure of the 1985 document. The guidelines also highlighted the differences in analysis for unsecured buildings in comparison to a building which has positive connections between floor, roof and wall elements, and cantilever elements secured or removed. Greater emphasis was placed on the assessment of the likely performance of URM buildings in their original form and with interim securing only in place, as distinct from the performance of the building with any strengthening work which was subsequently found to be necessary. Furthermore, material strengths were given in ultimate limit state format. Historic or heritage buildings were not given any specific or separate treatment, and the guidelines stated that:

> "the issues of risk versus the practicalities of strengthening associated with historic buildings require evaluation on a case-by-case basis. The principal problem with such buildings is that the greater the level of lateral forces that is specified for strengthening, the greater the risk of damaging the fabric that is to be preserved" (NZNSEE, 1995).

After the introduction of a new Building Act in 2004 (New Zealand Parliament, 2004) the Department of Building and Housing supported NZSEE in producing a set of guidelines, "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" (NZSEE, 2006). This was a major review and extension of previous guidelines, to account for the wider scope of the proposed new legislation. Prior to enacting The Building Act 2004, the term 'earthquake risk building' related only to URM buildings, but now an earthquake prone building could be of any material; steel, concrete, timber or masonry. The level of risk posed by buildings constructed as recently as the 1970s was more widely appreciated, in particular the inadequate performance of reinforced concrete structures due to deficient detailing. Definitions of "earthquake prone" and "earthquake risk" also changed. Essentially, earthquake prone buildings were defined as those with one-third or less of the capacity of a new building. While ~~The~~ Building Act itself still focussed on buildings of high risk (earthquake prone buildings), NZSEE considered earthquake risk buildings to be any building which is not capable of meeting the performance objectives and requirements set out in its guidelines, and earthquake prone buildings formed a subset of this. Moreover, NZSEE expressed a philosophical change, in acknowledgment of the wide range of options for improving the performance of structures that are found to have high earthquake risk. Some of these options involve only the removal or separation of components, and others affect a relatively small number of members. In line with performance-based design thinking, the term "strengthening" was replaced with "improving the structural performance of", highlighting the fact that such solutions as base isolation were not "strengthening" but were ~~an~~ effective way of improving structural performance.

The 2006 guidelines (NZSEE, 2006) provided both an initial evaluation procedure (IEP) and a detailed analysis procedure. The IEP can be used for a quick and preliminary evaluation of existing buildings, and takes into account the building form, natural period of vibration, critical structural weaknesses (vertical irregularity, horizontal irregularity, short columns and potential for building-to-building impact) and the design era of the building. Based on this analysis, if a territorial authority determines a building to be earthquake prone, the owner may then be required to take action to reduce or remove the danger, depending on the territorial authority's policy and associated timeline. The level required to reduce or remove the danger is not specified in The Building Act or its associated regulations. The Department of Building and Housing suggested that territorial authorities adopt as part of their policies that buildings be improved to a level "as near as is reasonably practical to that of a new building". Most territorial authorities took the view that they could not require strengthening beyond one-third of new building standard, but a significant number included requirements to strengthen to two-thirds of new building standard, in line with NZSEE recommendations. In developing policies on earthquake prone buildings, most territorial authorities recognised the need for special treatment and dialogue with owners when heritage buildings were affected. It is believed by the Department of Building and Housing that "the legislation has required each local community to put earthquake risk reduction on its agenda, and has left the local community to develop appropriate policies that reflect local conditions and perceptions of earthquake risk" (Hopkins et al., 2008).

The details discussed are summarised diagrammatically in Figure 1.5.

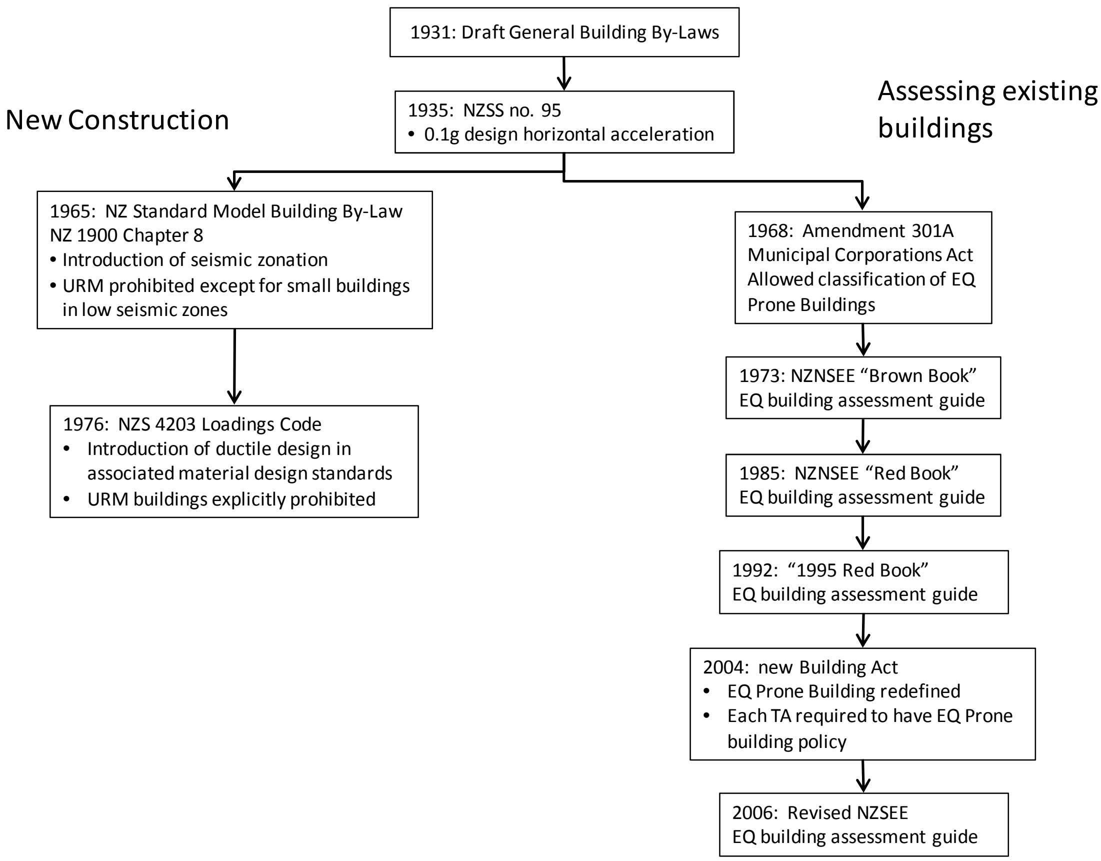


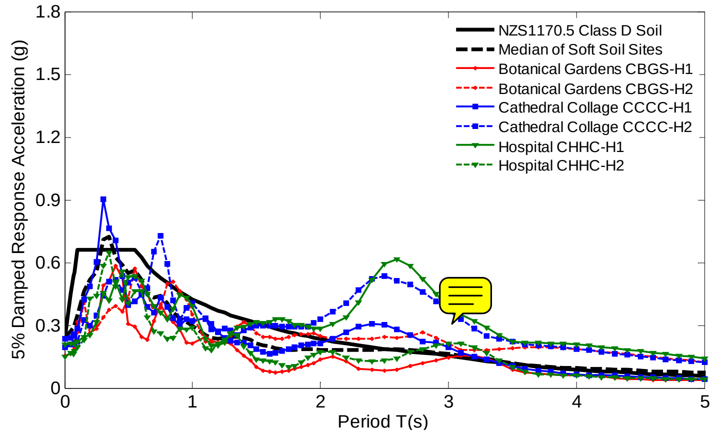
Figure 1.5 Flowchart showing evolution of New Zealand building codes and seismic assessment guides

1.4 Brief comments on the seismological characteristics of the 2010/2011 Canterbury earthquake swarm

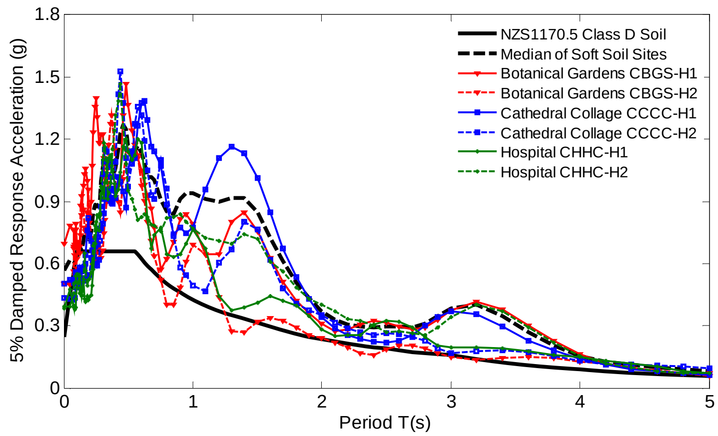
The brief seismological information presented below is provided primarily to illustrate the scale of the earthquake loading that was applied to the URM buildings stock of Christchurch and the surrounding areas with respect to the assessed seismic strength of these buildings and with respect to the design loading that was deemed appropriate for this region at the time of the earthquakes.

As at 23 July 2011 the Christchurch Quake Map website (http://www.christchurchquakemap.co.nz/) reports the location and magnitude of 3690 earthquakes/aftershocks that have occurred since 4 September 2010. Throughout this report this earthquake sequence is referred to as the '2010/2011 Canterbury earthquake swarm', with particular attention given to the two seismic events that resulted in the greatest deployment of resources associated with the collection of data on the performance of URM buildings, being the 4 September 2010 earthquake (referred to as the Darfield earthquake) and the 22 February 2011 earthquake (typically referred to as the Christchurch earthquake but sometimes referred to as the Lyttelton earthquake). It is acknowledged that there were additional events within the earthquake swarm that also caused damage to URM buildings, such as those on 26 December 2010 and on 13 June 2011. However, a study of the behaviour of URM buildings in the 4 September 2010 and 22 February 2011 earthquakes is deemed to be sufficient to convey an understanding of the overall impact of the 2010/2011 Canterbury earthquake swarm on the URM buildings located in Christchurch and the surrounding area.

As detailed in section 2.5 (see Figure 2.10 and Table 2.5), the assessed seismic capacity of all unretrofitted unreinforced masonry buildings in the Canterbury province was expected to be less than 67% of the New Building Standard (NBS), and furthermore approximately 40% of the Canterbury URM building stock was estimated to have a strength of less than 33%NBS. Unreinforced masonry buildings are comparatively stiff structures, with a fundamental period typically in the range of 0.3-0.5 seconds. From Figure 1.6(a) it can be established that for this period range many URM buildings were subjected on 4 September 2010 to earthquake loads that were between 67-100% of NBS (ie the solid line in Figure 1.6(a) corresponding to NZS 1170.5) and that the same buildings were subjected on 22 February 2011 to earthquake loads that were between 150-200% of NBS (see Figure 1.6(b). It is well established that URM buildings perform poorly in large earthquakes and consequently the level of earthquake damage observed in the Christchurch CBD is consistent with expectations for loading of this magnitude.



(a) Spectral accelerations recorded on 4 September 2010



(b) Spectral accelerations recorded on 22 February 2011

Figure 1.6 Earthquake spectral recording from the two principal earthquakes of the 2010/2100 earthquake swarm

Figure 1.7 shows a comparison of the median response recorded in the 4 September 2010 and 22 February 2011 earthquakes, clearly identifying that the February earthquake was far more severe in terms of the load that it applied to unreinforced masonry buildings (and all other buildings having a period of less than 2 seconds). Following the February earthquake a decision was made to increase the seismic zone factor Z to 0.3, and the effect of this modification is also plotted on Figure 1.7. The effect of this increase in the seismic zone factor was to increase seismic design forces and displacements by 36%.

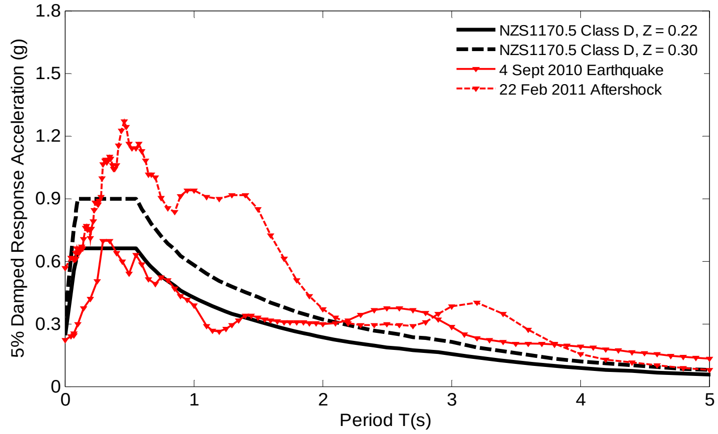


Figure 1.7 Comparison of earthquake spectra for the 4 September and 22 February earthquakes

Section 2:

The Architectural Characteristics and the Number and Seismic Vulnerability of Unreinforced Masonry Buildings in New Zealand

New Zealand's unreinforced masonry (URM) construction heritage is comparatively young, spanning from 1833 until approximately 1935 and peaking during the first four decades of the twentieth century. Consequently, a study of New Zealand's masonry building stock has a narrow scope in comparison with international norms (see Binda & Saisi, 2005; Lourenço, 2006; Magenes, 2006). This comparatively narrow time period has the advantage of facilitating the documentation and reporting of New Zealand URM construction practice with a greater degree of accuracy than is often possible in countries with an older and more diverse history of masonry construction (Binda, 2006).

2.1 Early Masonry Construction in New Zealand

Captain James Cook anchored off the coast of New Zealand on 9 October 1769. This event was followed by a gradual haphazard increase in the population of Europeans in New Zealand over the next 70 years. Jacobs (1985) reports that the European population of New Zealand in 1830 was probably a little more than 300, by 1839 the number had risen to possibly 2000, and at the beginning of the 1850s there were 26,000 Europeans in New Zealand. William Hobson's arrival in Auckland in 1840 as the First

Governor General of New Zealand marked the beginning of New Zealand as a British colony.

(a) 1866 View of the lower end, west side, of Queen Street, Auckland [Alexander Turnbull Library]

(b) Queen Street and Queen Street Wharf, Auckland, 1882 [Alexander Turnbull Library]

Figure 2.1 Early masonry construction in Auckland

Construction in Auckland in the period from 1840 to 1880 was primarily of timber for residential and small commercial buildings, but masonry buildings also began to appear close to the harbour (see Figure 2.1). Oliver (2006) reports that clay bricks were first manufactured in Auckland in 1852, with production of about 5,000 bricks per day. Timber was in plentiful supply and so it was only natural that outside the central city nearly all buildings were constructed of timber. Within Auckland central city the construction of timber buildings was not restricted until the City of Auckland Building Act of 1856. A fire in central Auckland in 1858 provided further impetus for the transition from timber to clay brick masonry construction.

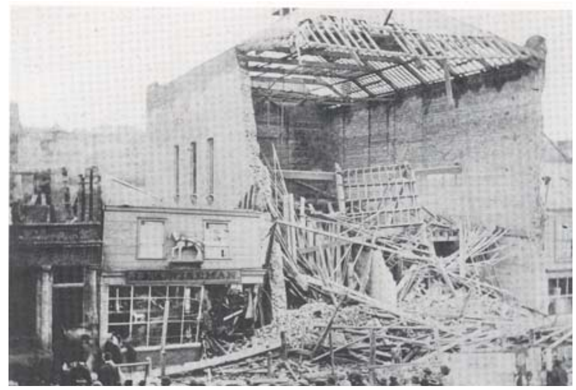
(a) The 1833 Stone Store at Kerikeri was built by the Church Missionary Society [Alexander Turnbull Library]

(b) Two Chinese miners in front of a stone cottage in central Otago, ca. 1860 [Alexander Turnbull Library]

Figure 2.2 Examples of early masonry construction in New Zealand

The lack of durable local building stone meant that the great majority of Auckland city's masonry buildings were constructed of clay brick with a stucco finish. In other parts of New Zealand there was a more plentiful supply of natural stone, with New Zealand's earliest masonry building having been constructed of stone in 1833 (see Figure 2.2(a)). Figure 2.2(b)[2] shows an example of early rural construction in parts of New Zealand were timber was scarce and natural stone was the primary construction material.

(a) **Looking down Shortland Crescent, Auckland, ca. 1865. Construction is a mix of timber, brick masonry and stone masonry [Alexander Turnbull Library].**



(b) **Collapse of a new masonry auction market building, Queen Street, 1865 [Alexander Turnbull Library]**

Figure 2.3 Transition from timber to masonry construction

Figure 2.3(a) shows Auckland at a time when the majority of buildings were constructed of timber, but a number of masonry buildings were becoming prominent. However Figure 2.3(b) shows that not all masonry buildings were well constructed. Hodgson (1992) reports that inferior materials and uncertain ground conditions were not uncommon in building projects of this period. Hodgson also reports that Auckland city went through a transformation during the 1870s when almost all timber buildings were replaced by masonry buildings. Figure 2.4 shows that by 1910 the central city was composed almost entirely of URM buildings.

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[2] Note that the style of unreinforced masonry construction shown in Figure 2.2(b) is not representative of the New Zealand URM building stock remaining today, and is not further considered in this report. Elsewhere in the world where this style of construction remains prevalent, past large earthquakes have repeatedly led to widespread and catastrophic collapse of this type of construction.

(a) **Looking along a row of commercial buildings on Queen Street, Auckland, ca. 1910 [Alexander Turnbull Library]**

(b) **Lorne Street, Auckland, ca. 1910 [Price Collection, Alexander Turnbull Library]**

Figure 2.4 Masonry building stock in Auckland in 1910

2.1.1 The influence of the Wairarapa and Murchison Earthquakes

The Wairarapa Earthquake occurred on Tuesday 23 January 1855 and had an estimated magnitude of M8.2 (Grapes & Downes, 1997). This earthquake is the largest to have occurred in New Zealand since the time of European colonisation (see Dowrick & Rhoades (1998) for a catalogue of major New Zealand earthquakes from 1901-1993). The shock was felt across almost the entire country, was highly destructive in Wellington, and also caused severe damage in Whanganui and Kaikoura.

(a) **General store damaged by the 1929 Murchison earthquake [Alexander Turnbull Library]**

(b) **Damaged business premises after the earthquake of 17 June 1929 [Alexander Turnbull Library]**

Figure 2.5 Damage to masonry buildings in the 1929 Murchison earthquake

The M7.8 earthquake that struck Murchison on the 17th of June 1929 was felt throughout New Zealand (Dowrick, 1994). Fortunately, the most intense shaking occurred in a mountainous and densely wooded area that was sparsely populated. Casualties were therefore comparatively light and the damage was mostly confined to the surrounding landscape, where the shaking triggered extensive landslides over thousands of square kilometres. Nonetheless, the shock impacted with damaging intensities as far away as Greymouth, Cape Farewell and Nelson (see Figure 2.5). Fifteen people were killed in the Murchison earthquake.

(a) Overlooking Napier City, ca. 1900 [Alexander Turnbull Library]

(b) Overlooking Napier at the buildings ruined by the 1931 earthquake and the fires [Alexander Turnbull Library]

(c) Hastings Street, Napier, ca. 1914 [Alexander Turnbull Library]

(d) View down Hastings Street, Napier after the earthquake 1931 [Alexander Turnbull Library]

Figure 2.6 Damage to masonry buildings in the 1931 Hawke's Bay earthquake

2.1.2 The 1931 Hawke's Bay Earthquake

As reported above, it was the combustibility of timber buildings that prompted the focus in Auckland towards building in clay brick unreinforced masonry, and occasionally in stone masonry. Early earthquakes in the Wellington region resulted in a slower adoption of masonry construction. This caution proved to be well justified. On the morning of 3 February 1931 the Hawke's Bay region of the eastern North Island was struck by a M7.8 earthquake that destroyed much of the city of Napier (see Figure 2.6). Fires swept through the wreckage, destroying much of what was left. Perhaps the largest brick masonry building to collapse was the Napier Anglican Cathedral (see

Figure 2.7). The shaking resulted in damage from Taupo to Wellington, and left 30,000 people homeless. The official death toll was 256, and the event currently remains the worst disaster of any type to occur on New Zealand soil (Dowrick, 1998; Dalley & McLean, 2005).

(a) St John's Anglican Cathedral in Napier, ca. 1885 [Alexander Turnbull Library]

(b) Ruins of the Napier Anglican Cathedral after the 1931 Hawke's Bay earthquake [Alexander Turnbull Library]

Figure 2.7 Napier Anglican Cathedral before and after the 1931 Hawke's Bay earthquake[3]

2.2 Architectural characterisation of New Zealand's URM building stock

In order to ascertain the structural seismic response of both individual URM buildings and the aggregated URM building stock, several key attributes of these building require characterisation. Within the characterisation of URM buildings, the broadest and most important classification is that of the overall building configuration. The seismic performance of an URM building depends on its general size and shape, as a small, low-rise, square building will behave differently when subjected to seismic forces than a long, row-type, multi-storey building. In addition to this, retrofit interventions which may be appropriate for one type of building may not be appropriate for another, different, type of building (Robinson & Bowman, 2000). Whilst a "one size fits all" approach is not viable for all URM buildings, for initial seismic assessments and vulnerability analyses, classification of buildings into typologies is a useful and necessary exercise. This exercise also enables a broad understanding of the financial and economic factors associated with seismic assessment and improvement of potentially earthquake-prone buildings.

The word typology is used as a classification according to a general type, and in the sphere of architectural characterisation different groupings of buildings can be classified

[3] Note the parallels to the damage observed to the Christchurch Cathedral as reported in section 5.1.1.

according to common features or elements. Tonks et al. (2007) began a preliminary identification of building typologies in New Zealand, based on those identified in Italy by Binda (2006). Three typologies were identified, differing from those identified in Italy because of age and materials:

- Stand alone isolated secular or religious buildings and chimneys;
- Row residential buildings;
- Row commercial and retail buildings.

It has since been identified (Russell, 2010) that the New Zealand building stock warrants seven typologies, which are outlined in Table 2.1, and photographic examples are given in Figure 2.8. Buildings are separated according to storey height, and whether they are isolated, stand-alone buildings or a row building made up of multiple residences joined together in the same overall structure. A suggestion for the expected importance level of the structure is also given, according to AS/NZS 1170.0:2002 (Standards New Zealand, 2002). All New Zealand URM buildings fall into importance level 2 or higher because of the number of people that can be expected to be in the building during or after an earthquake, with medium to high consequences for loss of human life. Within the identified typologies, further distinctions can be made. For example, Type A buildings can be divided into those which have a dividing wall down the centre (Type A1), and those which do not (Type A2). Type G buildings are generally monumental structures and those which do not fit easily into the other categories. Usually for such structures unique detailing is encountered, and unique analyses are necessary. Nevertheless there are useful sub-classifications which can also be made within this grouping. For example, Type G1 buildings are religious buildings and Type G2 are warehouses and factories with large tall sides and large open spaces inside. Further detail on each typology can be found in Russell & Ingham (2008).

Table 2.1 New Zealand URM typologies

Type	Description	Importance level (from NZS 1170.0)	Details
A	One storey, isolated	2, 4	One storey URM buildings. Examples include convenience stores in suburban areas, and small offices in a rural town.
B	One storey, row	2, 4	One storey URM buildings with multiple occupancies, joined with common walls in a row. Typical in main commercial districts, especially along the main street in a small town.
C	Two storey, isolated	2, 4	Two storey URM buildings, often with an open front. Examples include small cinemas, a professional office in a rural town and post offices.
D	Two storey, row	2, 4	Two storey URM buildings with multiple occupancies, joined with common walls in a row. Typical in commercial districts.
E	Three+ storey, isolated	2, 4	Three + storey URM buildings, for example office buildings in older parts of Auckland and Wellington.
F	Three+ storey, row	2, 4	Three + storey URM buildings with multiple occupancies, joined with common walls in a row. Typical in industrial districts, especially close to a port (or historic port).
G	Institutional, Religious, Industrial	2, 3, 4	Churches (with steeples, bell towers etc), water towers, chimneys, warehouses. Prevalent throughout New Zealand.

Typology A building – one storey isolated

Typology B building – one storey row

Typology C building – two storey isolated

Typology D building – two storey row

Typology E building – three+ storey isolated

Typology F building – three+ storey row

Figure 2.8 Photographic examples of New Zealand URM typologies (figure continues on next page)

Typology G building – religious

Typology G building – institutional

Figure 2.8 Photographic examples of New Zealand URM typologies

2.2.1 Parameters for Differentiating Typologies

Storey Height

URM building typologies are separated according to whether the buildings are one storey, two storey, or three or more storeys tall. While one and two storey buildings are approximately evenly distributed throughout the country, three and higher storey buildings are few in number and a single typology to classify all such buildings is sufficient. Buildings taller than three storeys are mainly located in the central business districts (CBD) of some of the largest cities, particularly Auckland, Wellington and Dunedin, as well as some port towns such as Timaru and Lyttleton in the South Island. Moreover, the difference in expected seismic behaviour between a three and four storey building is less significant than the difference between a one and two storey building. This comparative similarity is because three and higher storey buildings tend to be of masonry frame construction (on at least one face of the building, usually the front and back faces), in contrast to solid (with no window piercings) wall construction. As a broad generalisation, rocking of piers between windows and openings is the expected in-plane behaviour in masonry frames when subjected to lateral seismic forces (Abrams, 2000), and diagonal shear failure is less likely. For walls without openings (or with small openings), and depending on the magnitude of axial load, the expected in-plane failure mode in an earthquake is likely to be either sliding shear failure, diagonal tension (shear) failure, or rocking of the wall itself.

Building Footprint

The second primary characteristic for separating buildings into typologies is the building footprint, which differentiates buildings based upon whether they are a stand-alone, isolated, (almost) square building, or a row building made up of multiple residences joined together with common walls. This differentiation accounts for Typologies A – F, whereas those buildings with a non-uniform ground footprint (for example, many URM churches) will fit into the Typology G classification. In row structures containing walls that are common between residences, pounding has the potential to cause collapse, especially when floor or ceiling diaphragms in adjacent residences are misaligned. Different heights for the lateral force transfer into the common wall can result in punching shear failure of the wall, or diaphragm detachment and collapse. The effects of pounding are greater in the presence of concrete floor diaphragms, compared with timber diaphragms. Conversely in the case of many residences of similar height within the building, the seismic resistance is greatly enhanced due to the increased stiffness in one direction. Essentially square buildings with well distributed walls generally have a greater torsional resistance than buildings with less evenly distributed lateral force resisting walls (Robinson & Bowman, 2000) and long row buildings have different torsional properties than isolated buildings. A significant difference between isolated and row buildings becomes evident at the time of upgrading the building. An isolated building usually contains few residences, perhaps two shops for example. Row buildings may contain many residents, even ten or more. An isolated building is generally considered just that – a single building, whereas a row building, despite behaving in an earthquake as a single interconnected building, may be perceived as different buildings because it has multiple owners. It may be more difficult to perform remedial work on an entire row building at one time compared with retrofit of an isolated building. If retrofit interventions are implemented on only a part of a building, such an intervention may be ineffective.

2.3 New Zealand URM building population and distribution

Two independent methods with different primary data sources were used to estimate the number of URM buildings in existence throughout New Zealand in 2009. Data from Auckland City Council, Wellington City Council and Christchurch City Council, in conjunction with historic population data, were utilised to determine the distribution of URM buildings throughout the country and their associated construction dates (see Appendix B). In order to establish the financial value of existing URM buildings, data provided from Quotable Value New Zealand Ltd (QV Ltd) were used. This latter method also provided an estimate of the number of URM buildings. The validity of each approach was confirmed by their close agreement to determine the overall aggregate number of URM buildings in existence in New Zealand. The first method suggested that there were 3867 URM buildings in New Zealand (see Table 2.2), while the second method suggested that there are 3589 URM buildings (see Table 2.3). Taking the mean of both values indicates that there were approximately 3750 URM buildings in total existing in New Zealand in mid-2010[4].

[4] The reported analyses to determine the approximate number of URM buildings in New Zealand was performed prior to the 4th September 2010 Darfield earthquake. Recognising both the continual slow demolition of URM buildings nationwide and more recently the rapid number of URM buildings demolished in Christchurch, it was determined that the presented analyses were sufficiently accurate for the purpose of this exercise.

Table 2.2 Estimated provincial populations and number of URM buildings (see Appendix B for further details)

Province		Pre-1900	1901-1910	1911-1920	1921-1930	1931-1940	Total
Auckland	Population	175,938	193,581	278,357	393,639	516,886	
	URM	16	55	40	737	178	1026
Taranaki	Population	34,486	45,973	48,546	63,273	76,968	
	URM	3	11	7	118	25	164
Hawke's Bay	Population	37,139	46,906	51,569	65,037	77,652	
	URM	2	6	5	72	0	85
Wellington	Population	132,420	189,481	199,094	261,151	316,446	
	URM	27	127	169	243	111	677
Marlborough	Population	13,499	15,177	15,985	18,053	19,149	
	URM	1	3	2	27	6	39
Nelson	Population	33,142	45,493	48,463	49,153	59,481	
	URM	3	10	7	91	19	130
Westland	Population	15,042	15,194	15,714	14,655	18,676	
	URM	1	3	2	27	6	39
Canterbury	Population	145,058	166,257	173,443	206,462	234,399	
	URM	7	190	211	233	211	852
Otago and Southland	Population	174,664	156,668	191,130	206,835	224,069	
	URM	8	179	233	233	202	855
Total URM Building population by decade		68	584	676	1781	758	3867

2.4 Value of the New Zealand URM building stock

Table 2.3 summarises the number, total value and average value of URM buildings according to storey height. In the QV database the Building Floor Area and the Building Site Cover are recorded, and an estimate of the number of storeys can be obtained by dividing the Building Floor Area by the Building Site Cover, as the number of storeys is not directly recorded.

Table 2.3 URM building stock according to storey height[5]

Height	Number	Total Value	Average Value
1 storey	2526	$778,000,000	$308,000
2 storey	564	$256,000,000	$454,000
3 storey	163	$134,000,000	$822,000
4 storey	46	$54,000,000	$1,171,000
5+ storey	18	$20,000,000	$1,108,000
N/A	272	$259,000,000	$953,000
Total	3589	$1,501,000,000	

[5] All data entries were revised between July 2005 and September 2008, and all buildings are valued in New Zealand Dollars (NZ$) as at the date of valuation.

The Building Floor Area is the useable floor area and does not include the roof area. In some entries, either the Building Floor Area or the Building Site Cover is not recorded, and in this case the number of storeys is shown as N/A.

To put the New Zealand URM building stock in the context of the overall New Zealand building stock, the floor area provides a useful tool. A report prepared for the Department of Internal Affairs in 2002 (Hopkins, 2002; Hopkins & Stuart, 2003) showed that the total floor area of buildings in 32 cities and towns throughout New Zealand was approximately 27,200,000 m^2. The total floor area of URM buildings extracted from the QV database was approximately 2,100,000 m^2, suggesting that URM buildings make up approximately 8% of the total New Zealand commercial building stock in terms of floor area.

Figure 2.9 Number of URM buildings according to storey height

As shown in Table 2.3, New Zealand has in existence nearly 3600 URM buildings, with a collective financial value (in 2009) of approximately NZ$1.5 billion. The majority of the URM building stock consists of one-storey buildings, with the caveat on how this was determined noted above. It is clear from Table 2.3 that as the building height increases, the average value of the building also increases. Because the number of one-storey buildings is by far the greatest, the aggregate value of that building height is also the greatest, despite the comparatively low average value of each building. Thus it appears that the New Zealand URM building stock is largely made up of smaller, lower value buildings, and that in particular, the combination of one- and two-storey URM buildings constitutes 86% of the entire New Zealand URM building stock (see Figure 2.9). One-storey buildings make up 70% of all buildings, but only 51% of the total value of all URM buildings, and conversely buildings taller than one-storey make up only 30% of the number of buildings, but 49% of the value.

The average value of the building should determine the investment associated with seismic assessment and retrofit, and thus it may be concluded that while there are comparatively fewer larger buildings, the investment associated with their seismic assessment and retrofit can be justifiably higher. Similarly, low-rise buildings may require simplified and repeatable assessment methods and retrofit interventions.

Finally, it must be recognised that many buildings have a worth greater than their financial valuation, including an architectural, historic or heritage value to the community, which can be difficult to quantify (Goodwin, 2008; Goodwin et al., 2009).

2.5 Seismic Vulnerability of the New Zealand URM Building stock

Following determination of the number of URM buildings and their approximate regional distribution, the analysis was extended to determine the expected vulnerability of the URM building population. As part of the NZSEE Guidelines "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" (NZSEE, 2006), an initial evaluation procedure (IEP) is provided as a coarse screening method for determining a building's expected performance in an earthquake. The purpose of the IEP is to make an initial assessment of the performance of an existing building against the standard required for a new building, i.e., to determine the "Percentage New Building Standard" (%NBS). A %NBS of 33 or less means that the building is assessed as potentially earthquake prone in terms of the Building Act (New Zealand Parliament, 2004) and a more detailed evaluation will then typically be required. A %NBS of greater than 33 means that the building is regarded as outside the requirements of the Act, and no further action will be required by law, although it may still be considered as representing an unacceptable risk and seismic improvement may still be recommended (defined by NZSEE as potentially "earthquake risk"). A %NBS of 67 or greater means that the building is not considered to be a significant earthquake risk. NZSEE (2006) notes that:

> "A %NBS of 33 or less should only be taken as an indication that the building is potentially earthquake prone and a detailed assessment may well show that a higher level of performance is achievable. The slight skewing of the IEP towards conservatism should give confidence that a building assessed as having a %NBS greater than 33 by the IEP is unlikely to be shown, by later detailed assessment, to be earthquake prone" (see NZSEE (2006), chap. 3).

In collaboration with Auckland City Council during 2008, 58 buildings in Auckland City were assessed using the IEP. The %NBS of a building is determined by multiplying the "Performance Achievement Ratio" (PAR) (see NZSEE (2006) for details) by the Baseline $\%NBS_b$. For determining the $\%NBS_b$ for URM buildings, the following assumptions can reasonably be made in the context of the IEP (see Stevens & Wheeler, 2008):

- The construction date is pre-1935
- The period $T \leq 0.4$s
- The ductility factor, $\mu = 1.5$
- Most URM buildings have an importance level 2
- "Very soft soils" can be excluded.

Taking these assumptions into account, the only factor in determining the $\%NBS_b$ which varies between provinces is the seismicity at the site where the building is located. This is determined by the Hazard Factor, Z, which for each province was evaluated by averaging the Hazard Factors from the locations in that province (see Standards New Zealand, 2004). The PAR is a measure of an individual building's expected performance, independent of location, and primarily takes into account critical structural weaknesses, such as plan and vertical irregularity and pounding potential. It was determined from the analysis of the 58 buildings that the distribution of PARs in the sample was approximately normally distributed with a mean (x̄) of 1.6 and standard deviation (s) of 0.41. If it assumed that the PAR of all URM buildings in the country is also normally distributed, with the same mean and standard deviation as calculated for the sample population in Auckland City, the distribution of %NBS for all URM buildings in each former province in New Zealand can be estimated as follows:

$$s_{\%NBS} = \%NBS_b \times s_{PAR}$$

$$\bar{x}_{\%NBS} = \%NBS_b \times \bar{x}_{PAR}$$

For each province the Hazard Factor, $\%NBS_b$, and mean and standard deviation %NBS are shown in Table 2.4.

Table 2.4 Baseline $\%NBS_b$ for provinces

Province	Z	$\%NBS_b$	$\bar{x}_{(\%NBS)}$	$s_{(\%NBS)}$
Auckland	0.13	37.5	60.0	15.4
Taranaki	0.22	22.7	36.3	9.3
Hawke's Bay	0.39	12.7	20.3	5.2
Wellington	0.40	15.2	24.3	6.2
Marlborough	0.32	15.5	24.8	6.4
Nelson	0.27	18.0	28.8	7.4
Westland	0.34	14.5	23.2	5.9
Canterbury	0.22	22.1	35.4	9.1
Otago and Southland	0.15	32.5	52.0	13.3

Applying the mean number of URM buildings estimated from both analysis methods discussed in section 27 (3750 URM buildings in total) to the normal distribution of %NBS scores, an estimate of all the %NBS scores for each of the provinces can be evaluated, as shown in Figure 2.10. From Figure 2.10 the number of URM buildings in each province with an estimated %NBS below 33, between 33 and 67, and above 67 can be evaluated. Thus the number of URM buildings in each province which are potentially earthquake prone, potentially earthquake risk and unlikely to be significant, respectively, can be estimated. This data is shown in Table 2.5 and aggregated to determine the estimated overall number of URM buildings in these categories throughout all New Zealand, as shown in Figure 2.11. From these results (Figure 2.10, Figure 2.11, and Table 2.5), it can be seen that up to 35% of URM buildings currently existing in New Zealand could be potentially earthquake prone, and additionally up to 52% could be potentially earthquake risk, such that approximately only 13% of existing URM buildings can be expected to not be a significant earthquake risk. Most of these buildings are in regions of higher seismicity, which is the most critical factor in the vulnerability of URM buildings. Bothara et al. (2008) noted from assessments conducted in Wellington, that "most unreinforced masonry buildings have been confirmed as potentially earthquake prone." This statement is in agreement with the results presented here, in which 92% of URM buildings located in Wellington are estimated to be potentially earthquake prone.

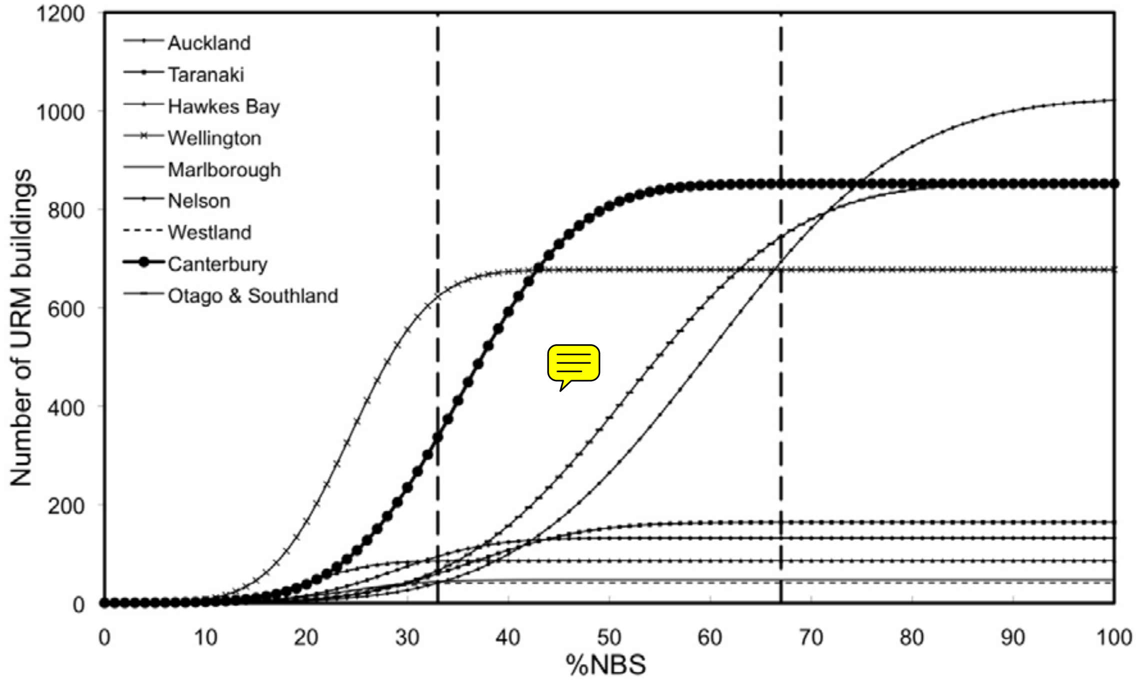


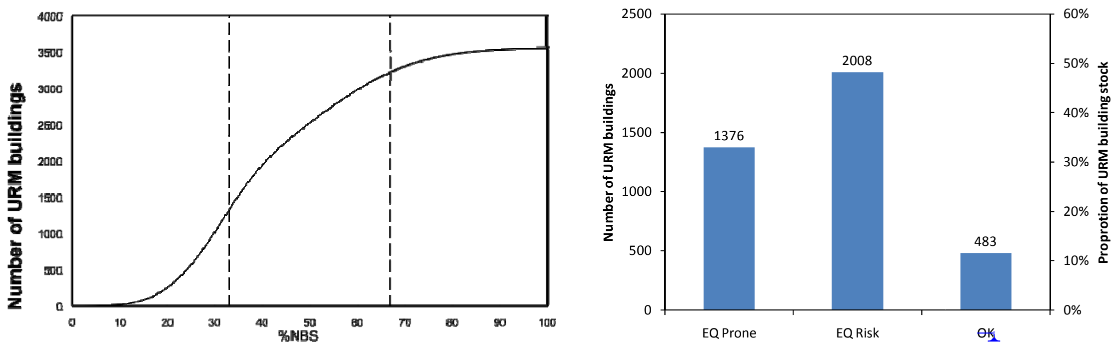
Figure 2.10 Estimated %NBS of URM buildings in Provinces throughout New Zealand

Additionally, 52% of all New Zealand URM buildings are estimated as being not earthquake prone as defined by The Building Act 2004, but can be expected to perform at a level less than 67% of the standard of a new building. NZSEE recommends that buildings with < 67%NBS should be seriously considered for improvement of their structural seismic performance. Thus up to 87% of all URM buildings in New Zealand could ~~require~~ seismic improvement, according to the ~~criteria~~ set by NZSEE (2006).

Table 2.5 Estimated number of potentially earthquake prone and earthquake risk URM buildings

Province	Potentially earthquake prone		Potentially earthquake risk		Unlikely to be significant risk	
Auckland	41	3%	628	31%	357	74%
Taranaki	59	4%	105	5%	0	0%
Hawke's Bay	84	6%	1	0%	0	0%
Wellington	622	45%	55	3%	0	0%
Marlborough	35	3%	4	0%	0	0%
Nelson	93	7%	37	2%	0	0%
Westland	37	3%	2	0%	0	0%
Canterbury	339	24%	513	26%	0	0%
Otago and Southland	66	5%	663	33%	126	26%
Total	1376	36%	2008	52%	483	12%

It must be recognised that the analysis presented here is essentially qualitative in nature and can be expected to overestimate the number of poorly performing URM buildings, primarily because of the conservative nature of the IEP. Nevertheless, as an informative estimate of the nature of the vulnerability of New Zealand's URM building stock, this analysis is considered robust. Additionally, this analysis does not take into account the number of buildings which have already been seismically improved, which Thornton (2010) notes is not insignificant.



(a) Estimated %NBS of URM buildings in New Zealand

(b) National estimate of potentially earthquake prone and earthquake risk URM buildings

Figure 2.11 Estimated earthquake vulnerability of New Zealand's unreinforced masonry building stock

Section 3:

Observed performance of unreinforced masonry buildings in the 2010/2011 Canterbury earthquake swarm

As previously noted in section 1.4, there have been over 3690 earthquakes and aftershocks associated with what is referred to here as the '2010/2011 Canterbury earthquake swarm'. In this section, attention is specifically given to the damage caused by the 4th September 2010 'Darfield earthquake' (M7.1) and the 22nd February 2011 'Christchurch earthquake' (M6.3) to URM buildings within the Christchurch Central Business District (CBD), which is defined here as the area bounded by the four avenues (Bealey, Fitzgerald, Moorhouse and Deans) and Harper Avenue. Other experts will discuss the seismological aspects of these two earthquakes. However, for completeness it is noted that whilst the Darfield earthquake was greater in its ~~Richter~~ magnitude (M7.1), its epicentre was located much further away (approximately 40 km) from the Christchurch CBD than was the M6.3 'Christchurch earthquake' whose epicentre was only 10 km from the Christchurch CBD (refer to Figure 3.1).

3.1 Damage to URM buildings from the 4 September 2010 earthquake

Post-earthquake inspection of building performance led to 595 URM buildings being assessed. It is believed that the majority of un-assessed URM buildings were undamaged and were located outside the primary inspection zone associated with the CBD and arterial routes extending from the central city. General features of the 595 assessed URM buildings are reported in Figure 3.2, indicating that the majority of buildings were either 1 or 2 storey, consistent with prior findings by Russell & Ingham (2010) (see also Figure 2.9). Figure 3.2(c) shows that the most common occupancy type was commercial or office buildings, and hence the majority of buildings were unoccupied at the time of the 4 September 2010 earthquake, significantly contributing to the lack of direct earthquake fatalities. The survey forms contained a field to record the estimated gross floor area of the building, and thus the estimated building footprint could be determined once accounting for the number of stories (see Figure 3.2(b)). Unfortunately the data are incomplete as only 301 entries were recorded for the 595 separate buildings assessed. It is not possible to establish from the database whether individual entries belonged to a stand-alone or a row building.

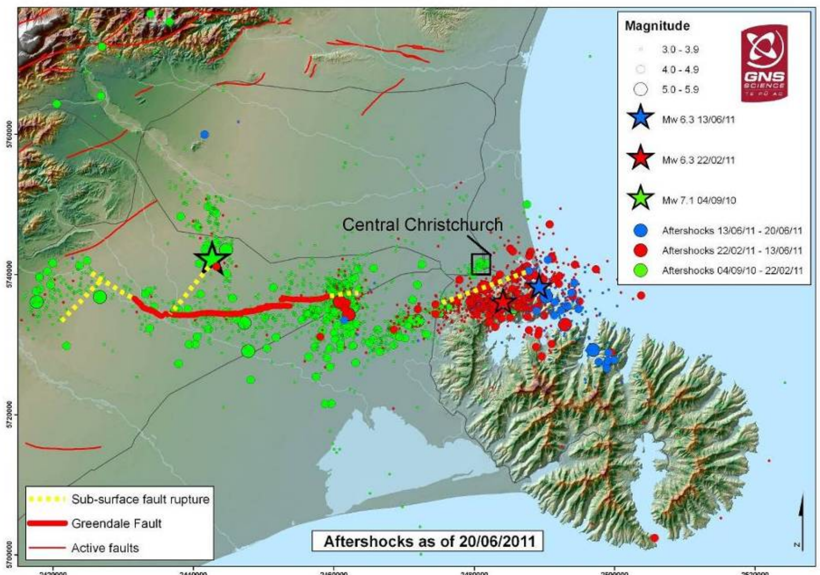
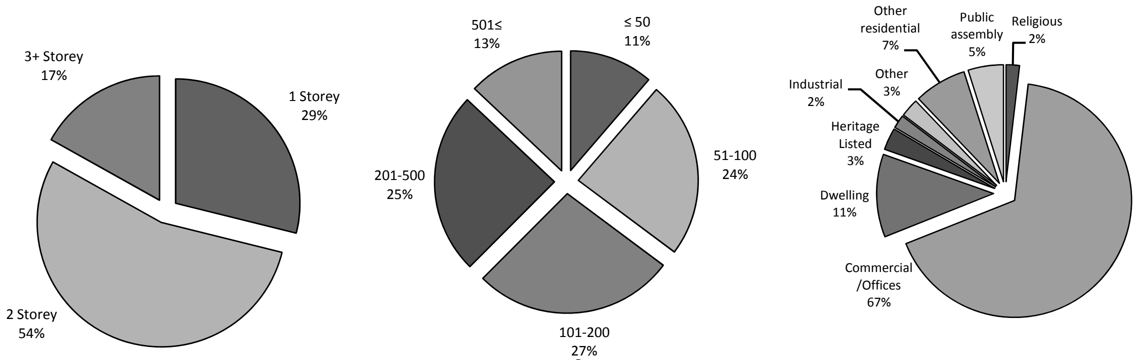


Figure 3.1 Epicentre locations for Sept 2010 and Feb 2011 earthquakes (from http://www.geonet.org.nz/canterbury-quakes/)



(a) Storey height (595 entries) **(b) Footprint area (m^2; 301 entries)** **(c) Occupancy type (595 entries)**

Figure 3.2 Building characteristics derived from interrogation of the inspection database (September 2010)

3.1.1 Material properties

The general observation from the debris of collapsed URM walls was that the kiln fired clay bricks were generally of sound condition, but that the mortar was in poor condition. In most cases the fallen debris had collapsed into individual bricks, rather than as larger chunks of masonry debris (refer to Figure 3.3(a)). When rubbing the mortar that was adhered to bricks it was routinely found that the mortar readily crumbled when subjected to finger pressure (refer Figure 3.3(b)), suggesting that the mortar compression strength was very low. However, it appears that superior mortar was often used in the ornate parapet above the centre of the wall facing the street, as this segment of the collapsed parapet often remained intact as the parapet collapsed (refer Figure 3.4).

(a) Masonry rubble showing 'clean' bricks **(b) Weak mortar crumbles between fingers**

Figure 3.3 Masonry rubble from collapsed wall

3.1.2 Building damage statistics

In general, the observed damage to URM buildings in the 2010 Darfield Earthquake was consistent with the expected seismic performance of this building form, and consistent with observed damage to URM buildings both in past New Zealand and Australian earthquakes and in numerous earthquakes from other countries. As part of the emergency response to this earthquake, the authors spent 72 hours assisting Christchurch City Council with building damage assessments, tagging buildings with either a green, yellow or red placard depending, respectively, upon whether a building was safe for public use, had limited accessibility for tenants/occupants, or was not accessible. Many examples of earthquake damage were observed during this exercise, as well as many examples of seismic retrofits to URM buildings that had performed well.

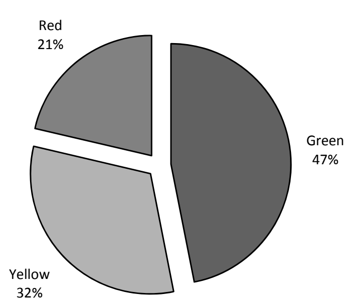
(a) Solid section of masonry gable

(b) Solid section of parapet

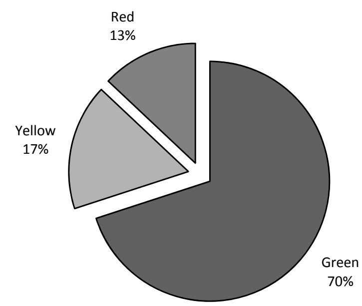
Figure 3.4 Large sections of masonry intact after fall from buildings

The results of the damage assessment are reported in Figure 3.5. Figure 3.5(a) reports the 'useability' assignment of the 595 URM buildings assessed. In consultation with staff of Christchurch City Council it was assumed that the remainder of the URM buildings thought to exist in Christchurch probably had a green tag usability rating, and so a theorised damage distribution for the entire URM building stock of Christchurch is shown in Figure 3.5(b).

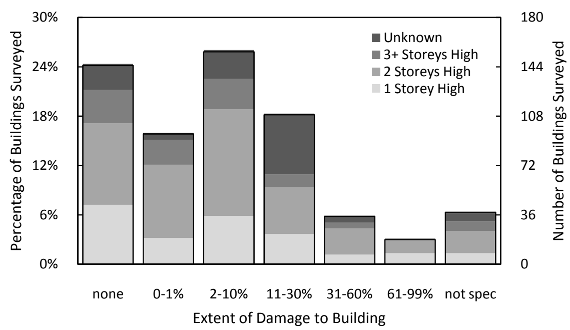
Figure 3.5(c) reports the level of damage in percentage terms for the 595 buildings that were surveyed by the Rapid Building Assessment teams. The values recorded by the teams for each building surveyed were simply estimates (excluding contents damage). Despite the known vulnerability of URM buildings to earthquake loading, 395 of the 595 buildings (66%) were rated as having 10% damage or less, with only 162 (34%) of the buildings assessed as having more than 10% damage. It was also possible to study the distribution of damage dependent on storey height (Figure 3.5(c)), with the data indicating no definitive trend and a comparatively uniform level of damage assigned to buildings in each height category.



(a) Distribution of placard assignments (595 entries)



(b) Theorised damage distribution for entire building stock (958 entries)



(c) Extent of building damage

Figure 3.5 Damage statistics for the 4 September 2010 earthquake

3.1.3 Chimneys

Unsupported or unreinforced brick chimneys performed poorly in the earthquake (Figure 3.6), with numerous chimney collapses occurring in domestic as well as small commercial buildings and some churches. Many examples of badly damaged chimneys that were precariously balanced on rooftops were also seen (Figure 3.6(b)) and it was reported that one week after the earthquake, 14,000 insurance claims involving chimney damage had been received, from a total of 50,000 claims (NewstalkZB, 2010). Emergency services personnel were in significant demand, being deployed to remove damaged chimneys in order to minimize further risk and eliminate these ‘falling hazards’ (Figure 3.6(c)). In contrast, Figure 3.6(d) shows an example of a braced chimney that performed well. Note that Figure 3.6(b) shows further evidence of the poor performance of mortar during the earthquake.

(a) Two damaged chimneys and gable wall

(b) Unstable damaged chimney

(c) Emergency service workers remove chimney

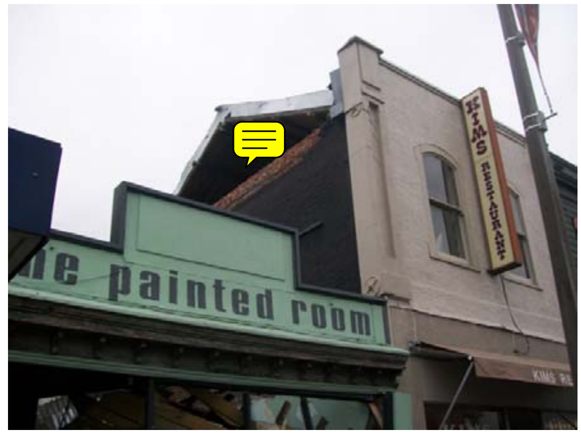
(d) Braced chimney performed well

Figure 3.6 Examples of chimney performance during the Darfield earthquake

3.1.4 Gable end wall failures

Many gable end wall failures were observed, often collapsing onto or through the roof of an adjacent building (refer to Figure 3.6(a) and Figure 3.7). However, there were also many gable ends that survived; many more than might have been expected, with the majority having some form of visible restraints that tied back to the roof structure. These examples are shown and discussed later (refer Figure 3.14 and Figure 3.15).

(a) 93 Manchester St



(b) 816 Colombo St

(c) Montreal-Armagh street corner

(d) Kilmore-Montreal street corner

Figure 3.7 Examples of gable end wall failures

3.1.5 Parapet failures

Numerous parapet failures were observed along both the building frontage and along their side walls. For several URM buildings located on the corners of intersections, the parapets collapsed on both perpendicular walls (refer Figure 3.8). Restraint of URM parapets against lateral loads has routinely been implemented since the 1940s, so whilst it is difficult to see these restraints unless roof access is available, it is believed that the majority of parapets that exhibited no damage in the earthquake were provided with suitable lateral restraint. In several cases, it appears that parapets were braced back to the perpendicular parapet, which proved unsuccessful.

(a) Multiple front wall parapet failures

(b) Corner of Sandyford and Colombo Street

(c) Side wall parapet collapse onto roof.

(d) Corner Columbo and Tuam Street

Figure 3.8 Examples of typical parapet failures

3.1.6 Anchorage failures

Falling parapets typically landed on awnings, resulting in an overloading of the braces that supported these awnings, leading to collapse. Most awning supports in Christchurch involved a tension rod tied back into the building through the front wall of the building. Many of these connections appear to consist of a long, roughly 25 mm diameter rod, with a rectangular steel plate (about 5 mm thick) at the wall end that is about 50 mm wide x 450 mm long and fastened to the rod and positioned either inside the brick wall or in the centre of a masonry pier or wall. In most cases the force on the rod exceeded the capacity of the masonry wall anchorage, causing a punching shear failure in the masonry wall identified by a crater in the masonry (refer Figure 3.9(a)).

(a) Anchorage failure

(b) Close-up of failed anchorage detail

Figure 3.9 Anchorage failure of awning brace due to parapet collapse

3.1.7 Wall failures

Out-of-plane wall failures were the first images to appear on television directly after the earthquake. Inspection of this damage typically indicated poor or no anchorage of the wall to its supporting timber diaphragm. Several examples of wall failure are shown below. Figure 3.10(a) shows a corner building that had walls fail in the out-of-plane direction in both perpendicular directions, on both sides of the corner. Figure 3.10(b) shows a 3-storey building where walls in the upper two stories suffered out-of-plane failures and Figure 3.10(c) shows similar damage for a 2-storey building. In all three of these instances, it appears that the walls were not carrying significant vertical gravity loads, other than their self weight, due to the fact that the remaining roof structures appeared to be mostly undamaged. In contrast, Figure 3.10(d) shows an out-of-plane failure of a side wall which was supporting the roof trusses prior to failure.

(a) Corner Worcester and Manchester streets **(b) 118 Manchester Street**

(c) 179 Victoria Street **(d) Failure of long wall**

Figure 3.10 Examples of out-of-plane failures in solid masonry walls

As shown in Figure 3.11, several examples of face load wall failure closely resembled observed damage in dry stack masonry experiments (Restrepo-Velez and Magenes, 2009), providing further support to the supposition that many of the wall failures were partly attributable to poor mortar strength.

(a) Wall damage at 140 Linchfield Street **(b) High speed photograph of a dry-stacked masonry wall failing during a tilt test**

Figure 3.11 Failure mechanism comparisons – observed earthquake damage versus experimental simulation

Cavity wall construction is generally believed to be much less common in New Zealand than is solid multi-leaf (or multi-wythe) construction. However, cavity wall construction can be extremely vulnerable to out-of-plane failure in earthquakes in situations where the cavity ties were poorly installed, or more commonly have corroded over time, as the wall is then comparatively slender and less stable than for solid construction. Figure 3.12(a) and (b) show examples of cavity wall buildings that suffered out-of-plane wall failures.

(a) Cavity wall failure in a residential building

(b) 832 Columbo Street

(c) Butterfly wall ties still intact

(d) Metal wall ties badly deformed.

Figure 3.12 Examples of out-of-plane failures in cavity walls

Figure 3.12(c) and (d) show that cavity ties were present but were insufficient to prevent the outer leaf from failing.

In some cases wall-diaphragm anchors remained visible in the diaphragm after the wall had failed, indicating that failure had occurred due to bed joint shear in the masonry (refer Figure 3.13(a)). Figure 3.13(b) shows a situation where a diaphragm anchor had been embedded within the wall. It can be seen that the anchor successfully prevented the restrained wall from failing, but was not able to prevent toppling of the parapet that was located above the anchor.

(a) Gable end failure despite anchor (see also Figure 13a).

(b) Wall anchor still intact (see also Figure 6a).

Figure 3.13 Wall-to-diaphragm anchor details

3.1.8 Successful wall anchorage

A significant feature of the earthquake was the number of occasions where anchored walls performed well during the earthquake. Photographs showing this are presented in Figure 3.14 and Figure 3.15. A typical wall-to-diaphragm (roof or floor) anchor typically consists of a long 20 mm bolt with a large circular disk of about 150-200 mm diameter between the wall exterior and nut that clamped the disk to the wall. This detail is shown quite clearly in Figure 3.13(a).

(a) Arts centre building

(b) Arts centre building

Figure 3.14 Successful gable end wall and side wall anchorages

3.1.9 In-plane wall ~~failures~~

Where walls exhibited some damage to in-plane deformation the cracks were mostly seen to pass vertically through the lintels over door or window openings. Although this type of damage was not widely observed, examples are shown in Figure 3.16.

(a) Front elevation

(b) Side elevation

Figure 3.15 Successful wall-floor and wall-roof diaphragm anchorages

(a) Extensive vertical cracking above window openings

(b) Vertical crack above window opening

(c) Vertical crack through spandrel

(d) Diagonal crack extending from window opening

Figure 3.16 Examples of in-plane wall damage above window openings

3.1.10 Partial wall failures

Another ~~interesting~~ feature of this earthquake is the observation of walls that only partly failed, allowing for identification of the specific failure mode at its onset. Several excellent examples are described below. The first of these is a 2-storey URM building on Ferry Road (see Figure 3.17) where the front, street facing, wall of the building had started to fail out-of-plane despite the presence of wall-roof diaphragm anchors. As is shown, the anchors were on the verge of pulling through the masonry wall. Internal inspection of the building revealed that the front wall had separated from the long side walls of the building and moved approximately 50 mm towards the road with respect to the ceiling/roof diaphragm (Figure 3.17(d)). It is believed that due to the nature of strength degradation of the brickwork at the onset of a punching shear failure, the anchorage has effectively failed and offers little residual resistance against further shaking. The only reason the wall did not completely collapse is probably due to the earthquake not imposing sufficient displacement on the wall after the anchorage failure.

(a) Building overview

(b) Detail of partial anchorage failure

(c) Onset of anchorage failure

(d) Internal view showing wall separation

Figure 3.17 Wall-roof anchorage failure and partial wall failure

A similar style of partial failure was observed in another building on Ferry Road (Figure 3.18(a)) but the authors were only able to observe the building externally. It should be noted that the buildings were not in close proximity to each other. An example of a gable end partial failure is shown in Figure 3.18(b), which can be compared to the ~~comparable~~ anchorage detail shown in Figure 3.13(a) that resulted in complete failure.

(a) Wall-roof anchorage failure

(b) Gable end anchorage failure

Figure 3.18 Partial bed joint shear failure surrounding anchorage detail

There were frequent examples of wall-diaphragm anchors that had deformed plastically. In these photographs (Figure 3.19), the circular plate can be seen to be slack due to plastic stretching of the anchor rod.

(a) Overview of wall anchors

(b) Close-up view of yielded anchor

Figure 3.19 Examples of yielded wall anchors

3.1.11 Diaphragm deformations

There was one instance where it was clear that diaphragm deformation, relative to the in-plane walls, contributed to partial failure of an out-of-plane wall. Figure 3.20 shows several views of a building which suffered out-of-plane parapet failure along its long, side walls. In Figure 3.20(b) it can be seen that the roof joists have tilted towards the front of the building. This suggested that the front wall of the building was driven forward at its top. Careful inspection of the front wall (Figure 3.20(c)) revealed a substantial outwards curvature which was most pronounced at the top.

(a) Overview of building

(b) Side-view of tilted joists

(c) Front wall curvature

Figure 3.20 Example of diaphragm deformation causing out-of-plane wall failure

3.1.12 Return wall separation

Many buildings exhibited substantial cracking between their front wall and side (return) walls. This damage is not necessarily a catastrophic problem if stiff horizontal diaphragms are well connected to the walls in both directions, but where there is not good diaphragm connectivity, there is the potential for complete out-of-plane collapse of one or both walls. Figure 3.21 shows some examples where major cracking was observed between the side return walls and the front parapet and wall.

Figure 3.21 Examples of wall separation at corners of buildings

3.1.13 Pounding

Several instances of damage due to buildings pounding against each other during the earthquake were observed. Figure 3.22 shows how the shorter building in the centre, which has different floor heights than the building to the left, damaged the column of the taller building at its top storey.

(a) Building overview **(b) Close-up of column** **(c) Close-up of column**

Figure 3.22 Example of building pounding damage

3.1.14 Special buildings

160 Manchester Street was a 7-storey office building that is reported to consist of load bearing masonry and was the most significant masonry building, at least in terms of height, in Christchurch (Figure 3.23). It ~~is~~ a registered heritage building and ~~is~~ a significant part of the fabric of the Christchurch city landscape. Unfortunately, the building suffered significant damage in the earthquake. The bottom two stories are reported to be reinforced concrete while the top five stories are reported to have load

bearing unreinforced masonry piers around the exterior of the building and a steel frame internally (columns spaced roughly at 5 m) with timber floors throughout (New Zealand Historic Places Trust, 2010). The masonry piers, having dimensions of approximately 1200 x 900 mm, were badly cracked at levels 3 and 4 (Figure 3.24). This damage was most likely due to the transition from concrete to masonry at level 3 and the fact that the adjoining 2-storey building located along the southern wall side stopped providing lateral support at that level. It appears that the lift core had received some strengthening previously, as well as the roof, perhaps in the late 1980s as reported by the New Zealand Historic Places Trust (2010). Close up photographs of the masonry piers at levels 3 and 4 show the primary damage that concerned the assessment teams (Figure 3.24). Further inspection by the assessment team exposed the internal face of one pier on the western face of the building to reveal that the external cracking continued through the entire pier thickness.

Figure 3.23 Manchester Courts building (view from NW)

(a) North wall piers, levels 3-4

(b) West wall piers, levels 4-5

Figure 3.24 Damage to masonry piers of Manchester Courts building at levels 3-5

Two days after the main earthquake, structural engineers met with Urban Search and Rescue Team leaders and city officials to determine a strategy for making the structure safe enough for building contractors and engineers to enter to determine more fully the extent of damage and the viability of repair. Four days after the main earthquake, the building had survived one M5.4 and three M5.1 aftershocks. After extensive deliberations the decision was taken to demolish the building.

St Elmo Court was also a 7-storey building that was ~~reported to be~~ a reinforced concrete frame building with external clay brick masonry piers. Owing to the absence of control joints between the masonry and concrete frame, it appeared that the masonry piers attracted sufficient seismic in-plane forces to cause shear failure (refer Figure 3.25). However, once the masonry cracked the seismic loads were transferred to the concrete frame. Judging by the extent of cracking in the brickwork, it appeared that the storey drifts developed during the 4 September 2010 earthquake were less than 1%, implying that the concrete frame was not pushed to its maximum capacity (strength or drift). ~~Following the 4 September 2010 earthquake the authors were not able to inspect the building from inside.~~

The building was demolished after the 22 February 2011 earthquake.

(a) Overview of building (b) Close-up of damage to brickwork

Figure 3.25 Views of St Elmo Court building, 47 Hereford Street

3.1.15 Building damage due to ground deformation

Perhaps the most striking aspect overall of the 2010/2011 Canterbury earthquake swarm was the extensive amount of liquefaction and ground deformation that occurred. These phenomena were not seen to a significant extent in the Christchurch CBD region containing the highest density of URM buildings, but did impact on a number of timber framed structures with masonry veneer. As shown in Figure 3.26, several cases of extreme ground deformation that affected URM buildings were observed outside of the CBD, and there were numerous cases where large crack widths formed in residential timber framed structures having a masonry veneer (Figure 3.26(c)). There were also cases where ground liquefaction had resulted in masonry structures having sunk into the ground (Figure 3.26(d)).

(a) Damage to masonry veneer due to ground deformation

(b) Wide cracks due to ground deformation

(c) Masonry veneer damage to recently built residence.

(d) Damage due to liquefaction

Figure 3.26 Damage to buildings having masonry veneer over timber frame, due to ground deformation and liquefaction

3.1.16 **Summary**

On the few occasions that building owners or occupants were in attendance it was possible to gain access to the interior of URM buildings and often observe that some separation had occurred between the floor and/or roof diaphragms and the masonry walls (in the out-of-plane direction). This damage was not easy to detect from the outside of a building, so that the damage reported from building surveys in the first 72 hours could be assumed to be a lower bound estimate of structural damage to URM buildings.

On the other hand, there were many instances of buildings that were structurally sound themselves but had suffered damage or were yellow or red-tagged owing to 'falling hazards' from neighbouring buildings. In some instances it was clear that a parapet or chimney from a neighbouring building had fallen onto or through the roof, being the only damage to the structure. In other instances, a building abutting a taller building with damaged parapet or gable side walls or chimney was given a yellow card (no public access) due only to the falling hazard posed by the structure next door. These examples of 'collateral damage and risk', such as that posed by 160 Manchester Street, and the associated business interruption costs, ~~make~~ the financial impact of this earthquake much greater than just the cost of rebuilding.

3.2 Damage to stone masonry buildings from the 22 February 2011 earthquake

Statistics regarding the damage to clay brick URM buildings from the 22 February 2011 Christchurch earthquake are still being compiled. It is expected that these statistics will be included in the final report to the Commission[6]. Consequently this section exclusively addresses unreinforced stone masonry buildings, including a comparison of damage reported following the 4 September 2010 and the 22 February 2011 earthquake.

The damage assessment inspections that were undertaken in September 2010 and again in April and May 2011 identified 90 unreinforced stone masonry buildings in Christchurch, many of which are included on the Historic Places Trust register of heritage buildings. Most of these stone masonry buildings were constructed between 1850 and 1930 and are masterpieces by important architects of the period, such as Benjamin Mountfort, Cecil Woods and John Goddard Collins, and are excellent examples of the Gothic Revival style. Significant examples include the Canterbury Provincial Council Buildings (see also section 5.1.3) and the former Canterbury University College, which is now referred to as the Christchurch Arts Centre (see also section 5.1.4). Besides their architectural value, these buildings represent the history of a relatively young country and for this reason resources should be directed towards their preservation and seismic improvement.

Most of the buildings considered in the study are now used for a variety of public functions, ranging from churches to public offices, schools and colleges, and incorporating both commercial and cultural activities.

The stone masonry buildings in Christchurch have similar characteristics both in terms of architectural features and in the details of their construction. This observation derives primarily from the fact that most of these structures were built over a comparatively short time period and were designed mostly by the same architects or architectural firms.

The vast majority of structures, and in particular those constructed in the Gothic Revival style, are characterized by structural peripheral masonry walls that may be connected, depending on the size of the building, to an internal frame structure constituted of cast

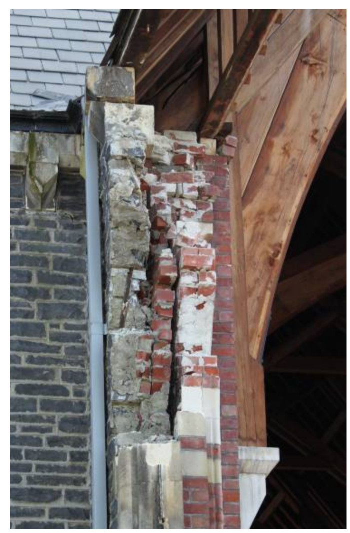
[6] Whilst final statistics are not available for damage to clay brick masonry buildings in earthquakes that occurred after 4 September 2010, it is clear from observations during the field survey work conducted since 22 February 2011 that the failure modes in later events were similar, with damage in the 22 February 2011 earthquake being both more prevalent and more severe in nature.

iron or steel columns and timber beams or to internal masonry walls that support flexible timber floor diaphragms and timber roof trusses. However, there are a few commercial buildings in the Christchurch CBD that are characterized by slender stone masonry piers in the front façade with the other perimeter walls constructed of multiple leaves of clay brick. These buildings are typically 2 or 3 stories in height, with 2 storey buildings being most common, and may be either stand alone or row buildings. The wall sections can be of different types:

- Three leaf masonry walls, with dressed or undressed basalt or lava flow stone units on the outer leaves (wythes) while the internal core consists of rubble masonry fill (Figure 3.27(a));
- Three leaf masonry walls, with the outer layers in Oamaru sandstone and with a poured concrete core, such as for the Catholic Cathedral of the Blessed Sacrament (Figure 3.27(b) and section 5.1.2);
- Two leaf walls, with the front façade layer being of dressed stone, either dressed basalt or bluestone blocks, or undressed lava flow units, and the back leaf constituted by one or two layers of clay bricks, usually with a common bond pattern, with the possible presence of a cavity or of poured concrete between the inner and outer leaves (Figure 3.27(c)).

(a) Cramner Court - 3 leaves with rubble fill.

(b) Cathedral of the Blessed Sacrament - Oamaru stone with poured concrete.

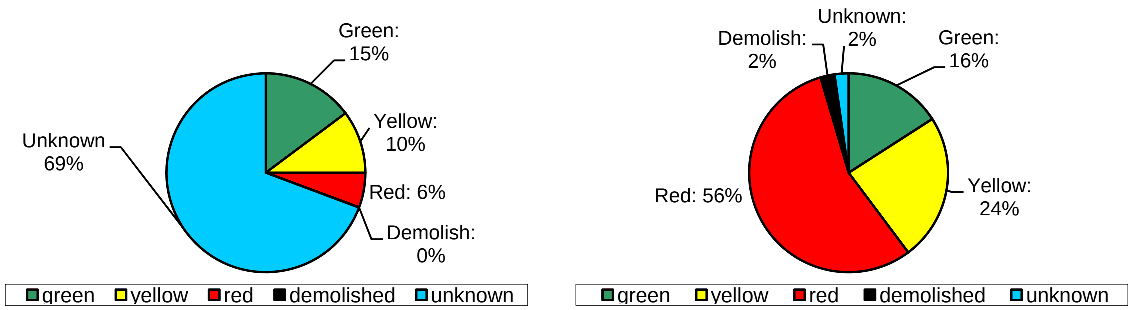


(c) St. Luke's Anglican Church - stone front façade with clay bricks back layers.

Figure 3.27 Representative examples of wall cross-sections for Christchurch stone masonry buildings

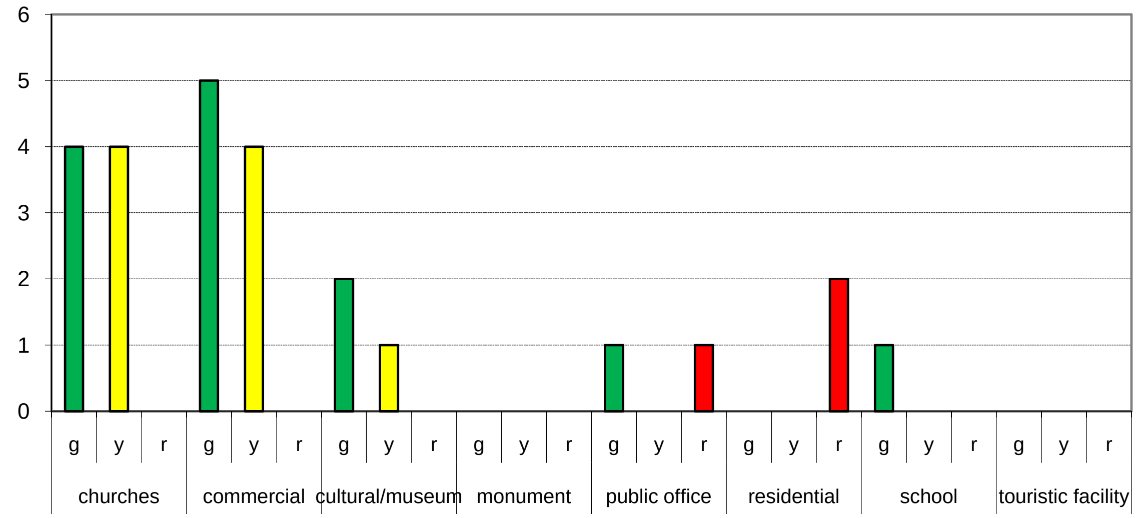
3.2.1 Post-earthquake assessment and building damage statistics

The seismic performance of stone masonry buildings was partially identified by considering the safety assessment data that was collected following the earthquakes that occurred in September 2010 and February 2011. Figure 3.28 shows the distribution of building safety assessments after the 4 September 2010 and 22 February 2011 earthquakes, respectively. From this figure it can be seen that there was a significant escalation of damage due to the continuing earthquake activity in the Christchurch region. Figure 3.29 gives a further breakdown of this data for the two major earthquakes on the basis of building usage. As noted earlier, green placards were assigned to structures that were deemed to be safe to re-enter and required no further intervention; yellow placards were applied to buildings whose accessibility was restricted due to minor damage; and red placards were applied to buildings that were considered unsafe and likely to have a moderate to severe level of damage. At the time of the study reported here, several buildings had been demolished already because of the hazard associated with their damage state. As shown in Figure 3.28, only 16% of the stone masonry buildings surveyed were assigned a green placard after the 22 February 2011 earthquake whereas approximately 50% (15% green compared with 16% yellow and red) had green placards after the 4 September 2010 earthquake.

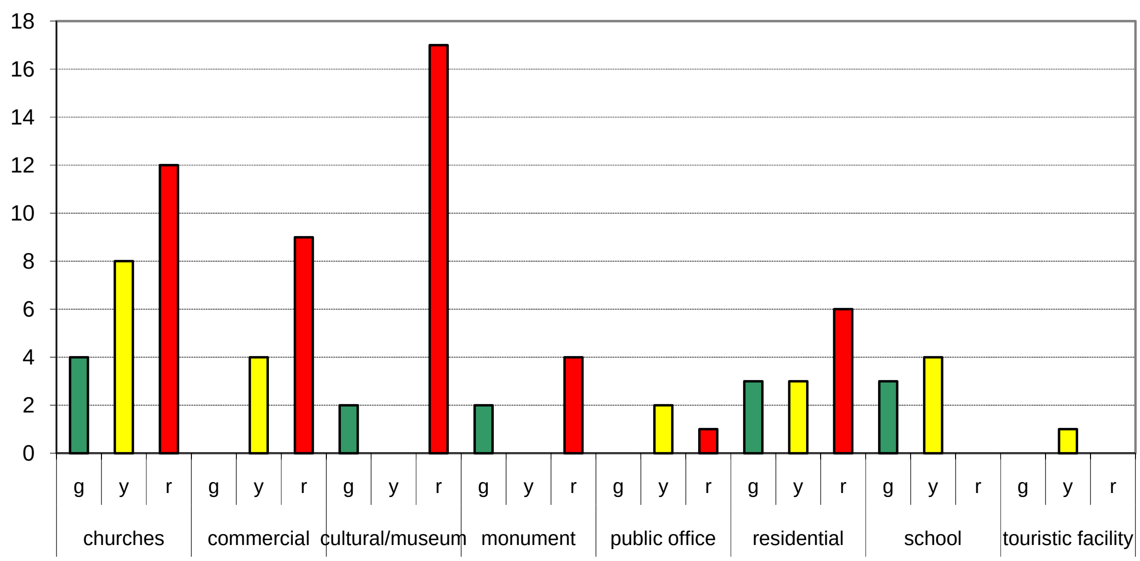


(a) After September 2010 **(b) Data updated 07 June 2011**

Figure 3.28: Distribution of safety evaluation placarding applied to stone masonry buildings

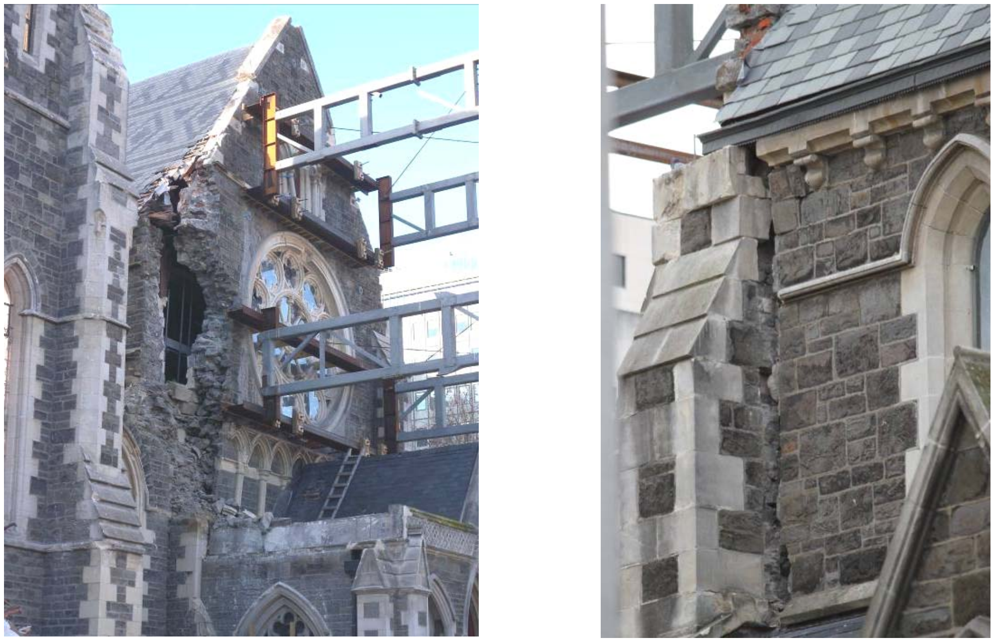


(a) after September 2010



(b) after February 2011

Figure 3.29 Distribution of safety evaluation placarding applied to stone masonry buildings differentiated by building usage



(a) unstable front wall (b) return wall separation

Figure 3.30 Christchurch Anglican Cathedral – front façade damage

3.2.2 Damage mechanisms in stone masonry buildings and churches

Many examples of earthquake induced damage mechanisms to stone masonry buildings were observed, with a detailed description of the most recurrent mechanisms presented below.

Out-of-plane failure mechanisms

As expected for buildings having architectural features typical of the Gothic Revival style (long span façades, flexible floor diaphragms and weak connections between walls), partial or global overturning or instability of the façades was reported for most of the structures inspected, with damage ranging from moderate to severe and in some cases reaching collapse. Examples are shown in Figure 3.30 to Figure 3.32 relative to the main façade of the Anglican Cathedral (now partially collapsed after the June 2011 earthquake and aftershocks), the Rockvilla dwelling that experienced complete collapse of the north and east façades, and the former Old Boy's High building in which the north façade was propped to avoid collapse due to out-of-plane failure. All of these buildings appeared to have poor connections between the walls at their corners, leading to return wall separation and subsequent out-of-plane failure of entire walls as in the case of the Rockvilla house (Figure 3.31).

Figure 3.31 Rockvilla dwelling with complete collapse of the north and east façades

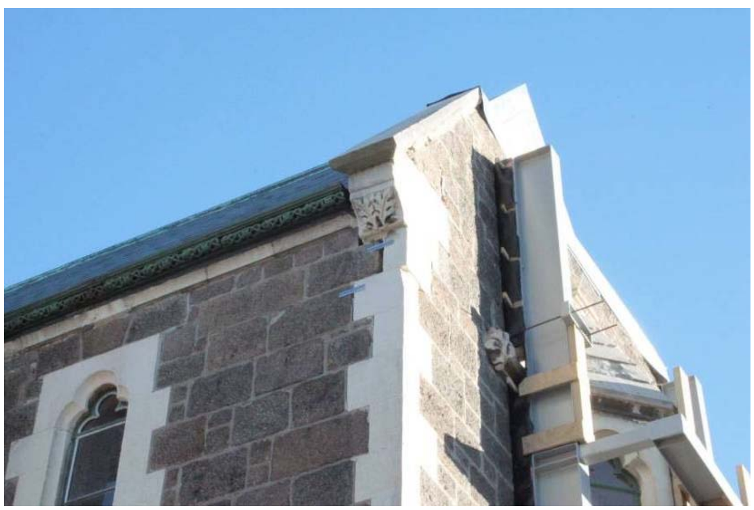


Figure 3.32 Christchurch Arts Centre (former Old Boy's High building), with severe damage due to instability of the façade at the second storey

Many of the buildings that were constructed in the Gothic Revival style sustained partial damage to their gable ends, with many cases of complete collapse of the gable. The absence of significant gravity loads and inadequate connection between the gable and roof trusses are primary contributing factors to this failure mode, along with increased accelerations experienced at the top levels of the structure (Figure 3.33).

Figure 3.33 Cramner Court, showing complete collapse of a gable

In-plane response of walls

Because the predominate direction of the 22 February 2011 earthquake was in the east-west direction, and because the buildings in the CBD are primarily oriented in the same direction, evidence of in-plane wall damage in the east-west running walls (see Figure 3.34 and Figure 3.35) was reported in conjunction with overturning of façades oriented in the orthogonal direction (see Figure 3.30).

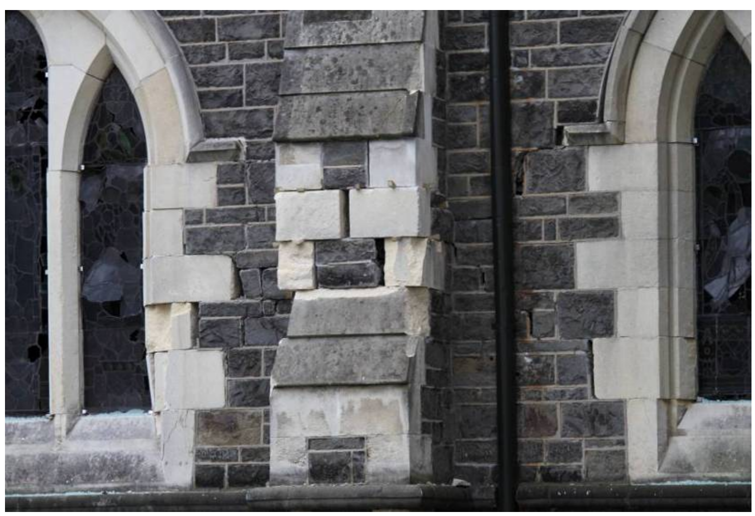
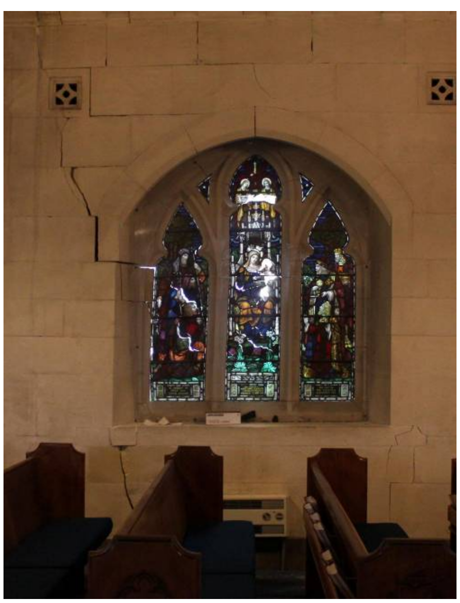


Figure 3.34 Christchurch Anglican Cathedral - diagonal cracks in the south façade piers

Figure 3.35 Canterbury Provincial Chambers - diagonal crack through entire south façade of the east annex

<u>Damage due to geometric irregularities</u>

Damage that was attributable to plan irregularity was frequently observed, particularly for stone churches, due to interaction between adjacent structural elements at the intersections between walls. In most churches where the bell tower or low annexes are connected to the nave, damage developed at the intersection of the different structures (see Figure 3.36 and Figure 3.37).



(a) Interior view

(b) Exterior view

Figure 3.36 St. Barnabas' Church, showing interaction between the nave and the bell tower

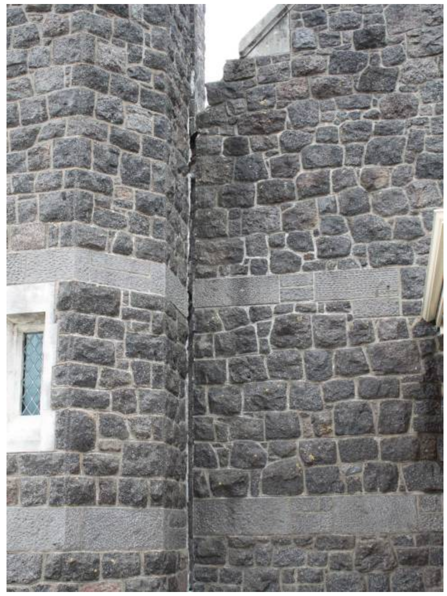


Figure 3.37 St. Mary's Anglican Church - detachment of the bell tower from the nave

Another distinct example of damage due to plan irregularity in association with differential foundation settlement was observed at the former Old Boy's High building. Figure 3.38 shows the vertical crack that formed at the intersection between two buildings constructed in successive phases, attributable to the lack of connectivity between the structural walls and their separate foundations.

(a) Distant view **(b) Close up view**

Figure 3.38 Interior views of Old Boy's High (part of the Arts Centre Complex, 2 Worcester St), showing interaction between adjacent buildings

Diaphragm and roof seismic response

The influence of both inadequate and adequate securing of walls and diaphragms using wall-diaphragm anchors was observed. In some cases anchors were either absent or were spaced too far apart to prevent bed joint shear failure of the masonry at the location of the anchorage. In those cases where anchoring had been seismically designed, or sufficiently closely spaced to resist lateral loads, the overturning of gables and other portions of walls was prevented.

(a) overturning of the front façade gable **(b) detail of failed wall-to-roof anchorage**

Figure 3.39 Former Trinity Church, showing details of gable ended out-of-plane wall failure

Two cases are presented to show the different behaviour induced by the presence and effectiveness of anchoring. Figure 3.39(a) shows the damage resulting from overturning of the gable of the main façade of the former Trinity Church in the Christchurch CBD while the detail in Figure 3.39(b) illustrates how the anchoring was insufficient in size and spacing to secure the wall in place. Figure 3.40 shows some examples of successful wall-to-roof anchoring in the Arts Centre building.

(a) former Old Girl's High **(b) former Canterbury Engineering Department**

Figure 3.40 The Christchurch Arts Centre, showing successful use of wall-diaphragm anchorages

In the case of churches, hammering of roof trusses was reported as for the case of St. James' Church shown in Figure 3.41.

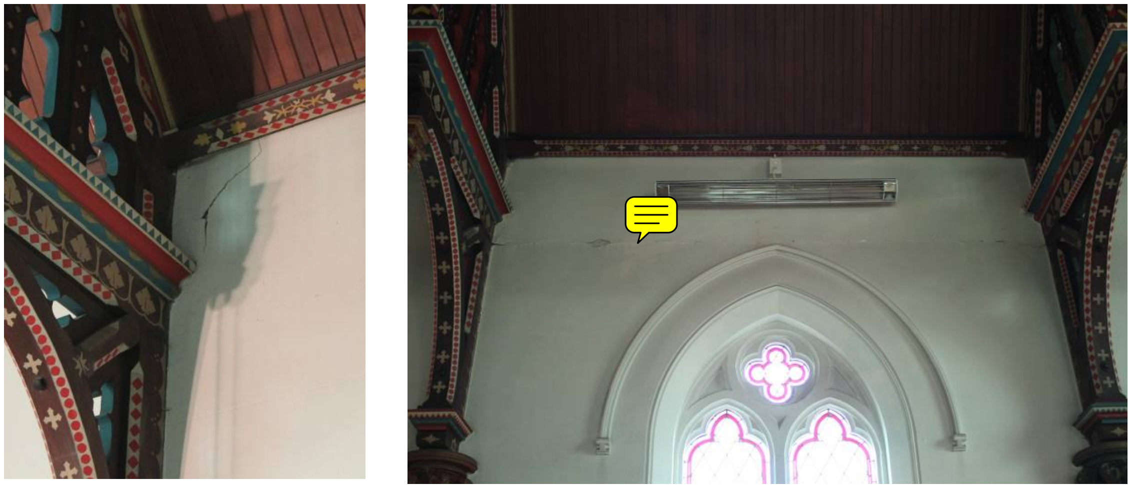


Figure 3.41 St James' Church, hammering of roofing elements on the walls of the nave

Damage induced by poor quality of construction materials

The quality of construction materials played a key role in the response of stone URM buildings. As previously described, one of the typical features of stone URM buildings in Christchurch is the different types of stone and mortar quality present in structures built with three-leaf walls. The use of soft limestone, such as Oamaru stone or the red tuff extracted in the Banks Peninsula, in conjunction with the use of low strength lime mortar, often lead to poor earthquake response. Examples of such behaviour include the Holy Trinity Church in Lyttelton, one of the oldest constructions in Canterbury, and St. John's the Baptist and the Time Ball station, as represented in Figure 3.42 to Figure 3.44.

Figure 3.42 Lyttelton Holy Trinity Church. Damage induced by hammering of the roof

Figure 3.43 St. John's the Baptist Church. Local collapse of material

~~It has been reported that after~~ the 13th June 2011 earthquakes, the remaining of these two buildings, and several others in Lyttelton that were in a similar state of damage, ~~completely~~ collapsed.

Figure 3.44 Time Ball Station. Damage in the Time Ball tower

Section 4:

Techniques for seismic improvement of unreinforced masonry buildings

The purpose of this section is to describe recognised techniques that are available for the seismic improvement of unreinforced masonry (URM) buildings. Typical failure modes are presented in Section 4.1 with reference to the observed performance of URM buildings in the 2010/2011 Canterbury earthquake swarm as documented in Section 3. A brief description of both well proven and recently developed techniques that have been implemented successfully in Christchurch for seismic improvement of URM buildings is presented in Section 4.2. Photographic evidence is provided to illustrate both successful and unsuccessful examples of retrofit techniques that had been installed in Christchurch URM buildings before the 4th September 2010 Darfield earthquake.

4.1 Typical earthquake failure modes in URM buildings

Decisions on whether to seismically retrofit a URM building or to demolish and rebuild a replacement structure that complies with current earthquake strength criteria depend upon the desired building performance as well as the associated costs. In this section, a generic retrofit strategy is described that begins with the most basic, and important, items to address with the primary aim of ensuring public safety. Additional retrofit measures may be taken beyond these to further improve building performance in order to minimise damage to the building and contents, with the highest performance target conceivably being to have the building and its contents suffer no damage and be immediately functional following the considered earthquake event.

Unmodified URM buildings usually have a number of inherent structural features ~~which~~ make them prone to earthquake forces. Many of these features can often be addressed without significant alteration to the building fabric, resulting in a relatively large increase in strength (Robinson & Bowman, 2000). The overarching problem is that New Zealand's URM building stock were simply not designed for earthquake loads, and whilst these buildings can be ~~made to perform adequately in an earthquake,~~ they lack a basic degree of connection between structural components to allow all parts of the building to act together. Therefore, the basic philosophy followed here is to first secure non-structural parts of URM buildings that represent falling hazards to the public (eg, chimneys and parapets) followed by improving the connections between the structural elements (roof, floors and walls), strengthening of specific structural elements, and possibly adding new structural components to provide extra support for the masonry building. In the rest of this section, the most commonly observed failure modes are described and possible retrofit strategies for each are given.

Chimney and parapet failures

Chimneys and parapets are parts of URM construction that project above the roof of the building. When subject to seismic actions, they act as cantilevers which rock on their supports at the roof line. If sufficiently accelerated by the earthquake, they will topple over (see Figure 3.6 and Figure 3.8). The simplest way to prevent earthquake failure of these elements is to brace them back into the roof structure (see Figure 3.6(d)). Implementation of this bracing is comparatively straightforward and inexpensive.

Gable end wall failures (missing or inadequate ties/anchorage)

Gable end walls sit at the top of walls at the end of buildings with pitched-roofs (refer to Figure 3.7). If this triangular portion of the wall is not adequately attached to the roof, the gable end section of the wall will rock as a cantilever (similar to a chimney or parapet) and is similarly vulnerable to outward collapse. An example of a building that was undergoing gable wall retrofit at the time of the February Lyttelton earthquake is shown in Figure 4.1 where the retrofitted gable walls had survived whereas the one gable wall remaining to be anchored to the roof truss failed. Other examples of gable end walls that performed poorly in the Canterbury earthquakes are shown in Figure 3.7 whilst examples of URM buildings that performed adequately due to the presence of anchor plate connections between the gable wall and the roof structure are shown in Figure 3.14.

Out-of-plane wall failures

Unreinforced masonry walls are weak in out-of-plane bending and therefore are susceptible to out-of-plane failures as shown previously (see Figure 3.10). The earthquake vulnerability of a URM wall to out-of-plane bending is predominantly dictated by its slenderness. Cavity walls (e.g. two single brick thick walls separated by a 75 mm gap that are connected by small metal ties) that are missing wall ties or have wall ties that are badly deteriorated are especially vulnerable (refer to Figure 3.12).

Solid walls can also be vulnerable but they have the advantage of being less slender. Examples were observed of out-of-plane failures of solid walls. The addition of wall-to-diaphragm anchors serves to reduce the vertical slenderness of a wall as well as make the building work together as a whole, rather than as independent parts.

Figure 4.1 Example of a secured gable end that survived earthquake loading and a companion failed gable end that was not secured

Floor and roof diaphragm failures (excessive deformation)

In some cases the floor and roof diaphragms, which are typically constructed of timber, were excessively flexible. This flexibility resulted in the walls that were connected to these diaphragms undergoing sufficiently large out-of-plane deflections to cause major wall damage and collapse. A number of successful diaphragm stiffening retrofits were observed, with details presented in the following section.

In-plane wall failures (piers and spandrels)

When out-of-plane failure mechanisms are prevented, the building is able to act as a complete entity and in-plane wall failure mechanisms can occur. It should be noted that when in this condition, building strength is often not far off the full design strength requirements. Strengthening of piers and spandrels can result in further increases in overall building strength. The seismic retrofit strategy for a building in this condition might be to improve the building's displacement capacity, rather than institute any further increase in strength. This intervention could be achieved by locally reinforcing the masonry spandrels and/or piers. Alternatively, ductile steel or concrete frames can be inserted internally to provide the in-plane shear strength needed, whilst also becoming responsible for some or all of the gravity load carrying function of the masonry walls. In effect, the introduction of a new internal structure converts the URM building into a frame structure with masonry veneer cladding.

Return wall separation

This failure mechanism (see Figure 3.21) is undesirable because it allows a wall over the entire building height to fall outwards. This failure mode can be ~~prevented~~ by the use of anchors installed along the vertical intersections between walls.

Pounding failures

This failure mechanism only occurs in row type construction where there is insufficient space between adjacent buildings so that they pound into each other when vibrating laterally during ~~an~~ earthquake. Widespread examples of pounding damage to URM buildings were observed in the recent Canterbury earthquakes (see Figure 3.22).

4.2 Techniques for seismic improvement of URM buildings

4.2.1 URM material stabilisation (poor maintenance)

Aim: Ongoing building maintenance should be undertaken to ensure that the masonry elements (walls, parapets, chimneys, and facades), and the timber roof and floor elements are in sound condition. Deterioration of the fundamental building elements compromises the ability of the 'as-is' connections between elements to share the seismic forces generated during an earthquake.

The bricks and particularly the mortar used in URM buildings deteriorate in the environment over time. Occasionally this deterioration will result in local failures and cracking which affect the overall effectiveness of the building. Various external actions such as dampness, subsidence, earthquakes, and impacts can also cause cracking and damage in the masonry elements. Deterioration similar to that shown in Figure 4.2 can often be remedied by reinstatement and repointing of mortar[7], but sometimes more substantial measures are required. There are various techniques for the repair of cracks, securing of lintels, and reinstatement of damage. Bonding agents such as grout or epoxy can be injected into the mortar and there are also several metal-based types of inserts, such as shaped dowels or reinforcing bars, which can be used to reinstate and strengthen the brickwork (Croci, 1998). The visual impact of reinstatement and strengthening can be minimal if done carefully, and the result is potentially far superior to a cracked and broken façade. However such measures are often irreversible, and care needs to be taken with colour matching and the concealment of holes drilled for inserting rods. Lintels and arches will sometimes require strengthening, particularly when these elements are constructed from URM. One of the best ways to achieve this intervention is by using drilled and inserted rods which are grouted or epoxy anchored into place. These rods provide the requisite tensile strength to the structural element while having little visual impact.

[7] Lime mortars should always be repointed with new lime mortars. Mixing lime and Portland cement mortars can cause numerous problems.

Figure 4.2 Severely degraded bricks and mortar due to moisture ingress

4.2.2 Parapets and other falling hazards

Aim: Secure or remove falling hazards. The greatest threat to public safety posed by URM buildings is that of falling masonry. This hazard can be due to chimneys that fail by rocking, usually at the roof line, and fall through the building's roof or over the side of the building. Parapets that are not properly secured to the building can fail similarly. Because of their location along the front and sides of commercial buildings, and because they typically fall outwards towards the footpath/street, parapets pose a very high danger to the public. Many of these failures were seen during both the 4 September 2010 and 22 February 2011 earthquakes, where parapets not only fell towards the street/footpath but they mostly fell onto the building's awning or canopy that projects above the pedestrian access, and resulted in collapse of that element as well. In cases of multi-storey (two or three) buildings with parapet failures, the parapets fell across the footpath and well into the street, crushing cars and buses and in several instances killing the occupants of those vehicles. Gable end walls are another version of this out-of-plane failure mechanism and similar to parapets, gable walls almost exclusively fall outwards. Where the gable walls are adjacent to public spaces, they also pose extreme danger to the public.

The basic strategy to eliminate these falling hazards is to fasten them to the rest of the structure, normally through use of ties or anchors back to the roof structure. Many examples of successful chimney, parapet and gable wall retrofits were observed.

URM buildings will often feature numerous decorative elements built with brick and plaster which are important parts of the building's architectural character, such as parapets, chimneys, gable walls, and other, smaller, decorative features. In the past, some buildings have had these elements removed wholesale, rather than the elements being strengthened or secured. Parapets and chimneys are usually the first parts of a building to fail in an earthquake due to their low bending strength and high imposed accelerations (FEMA 547, 2006). Parapets in particular are comparatively simple to strengthen. Generally a continuous steel section running horizontally along the length of the parapet which is fixed back to the roof structure behind is a suitable technique, if a little crude. The back of a parapet is not often seen, which makes the visual impact of this method low, and the steel section is bolted to the URM, which also has good potential for reversibility.

Several examples of unsuccessful parapet retrofits were observed following the recent Canterbury earthquakes. These failures provide an important opportunity to identify aspects that need to be considered when formulating best practice examples for use in future retrofit designs. Figure 4.3 shows two examples where discontinuous horizontal elements were installed at the rear of the parapet. In Figure 4.3(a) the distance between the braces securing the parapet to the roof structure was too large and in Figure 4.3(b) and (c) the horizontal element that was used to secure the parapet was discontinued adjacent to the corner of the building.

(a) Roof level view of failed parapet restraint

(b) Exterior view of failed parapet at corner

(c) Roof level view of failed parapet at corner

Figure 4.3 Failed parapet where the securing was discontinuous at the corner of the building

Equally important has been the widespread observation that many steel fixings that were installed inside URM buildings to internally secure gable walls and prevent out-of-plane wall failure have failed due to two companion failure modes:

- There has been a significant number of observed failures of adhesive anchors, where the anchor has withdrawn from the brick (see Figure 4.4(a)). This failure mode is of major significance as this securing technique has been used widely internationally. Recognising the significance of these observations, an international study between the University of Minnesota and the University of Auckland is currently underway in Christchurch to obtain reliable data on the pull-out strength of this class of anchor[8].

- There are many examples where the adhesive anchor has held the brick to which it was secured, but that brick has detached from the masonry structural element and only an individual brick is retained (see Figure 4.4(b)). This failure mode demonstrates the need for application of a continuous supplementary structural element to the surface of the masonry to secure the structural element as a single component.

(a) Failure of a steel fixing due to anchor withdrawal

(b) Failure of a steel fixing due to both anchor withdrawal and brick detachment

Figure 4.4 Examples showing failure of adhesive anchors

Chimneys contribute to the architectural form of a building and often help define its roofscape, and as such should be preserved if possible. The securing of chimneys is more complex than the securing of parapets and gables, but can usually be achieved by fixing them to the building diaphragms at each level and either strengthening the projecting portion or bracing it back to the roof structure with steel members similar to the methods used for parapet restraint, or fixing steel sections to the sides to provide flexural strength. A number of strengthening solutions are available for bonding to the surface of masonry elements and may be appropriate where the exterior has been plastered. Two such techniques used to strengthen chimneys are shown in Figure 4.5.

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[8] Professor Arturo Schultz from the University of Minnesota is the Principal Investigator of this project, with funding provided by the US national Science Foundation: Grant #CMMI-1138614, 'Data Collection on the Performance of Adhesive Anchor Retrofits in Unreinforced Masonry Buildings during the February 2011 Christchurch, New Zealand Earthquake'.

Other elements that constitute falling hazards, such as decorative plaster features on the face of a wall, can be effectively fixed with a single bolted connection. Less secure elements, such as plaster finials or balusters, can be secured with a single adhesive anchor connected to a strand of stainless steel wire, to mitigate the falling hazard. However, more complex strengthening work may be appropriate in some cases.

(a) Vertical Near Surface Mounted (NSM) Fibre Reinforced Polymer (FRP) strip strengthening of chimneys

(b) Fibre reinforced shotcrete applied to the exterior surface of a chimney

Figure 4.5 Examples of earthquake strengthened chimneys

4.2.3 Wall strengthening to restrain out-of-plane bending

Aim: Prevent out-of-plane failure of walls by increasing their flexural strength or reducing the vertical and horizontal distance between their supports.

URM walls are weak when subjected to forces other than compression. Even when fully secured to floors at each level, out-of-plane forces can cause significant wall bending that is governed by the ratio of the height between levels of support to the thickness of the wall (Derakhshan, 2011; Rutherford & Chekene, 1990). Some walls have sufficient thickness or have cross-walls or buttresses which enable them to withstand these out-of-plane forces without modification, however many walls will require seismic improvement. There are a number of approaches to combat this problem as described below.

Brick Cavity Walls – (Outer leaf fixing)

The outer leaf of a cavity wall is problematic as it is particularly susceptible to failure by peeling off outwards. The steel ties ~~which~~ were commonly installed to connect this layer to the more robust wall behind are subject to deterioration and sometimes missing, requiring attention during retrofits (Russell et al., 2006). One approach to this problem has been to fill the cavity with reinforcing steel and a cementitious grout, which has the dual benefits of bonding the outer leaf to the inner leaf and also forming a reasonably strong shear wall which is hidden from view. However, this approach fails to consider the purpose of a ventilated, drainable cavity. When a cavity is filled, not only is the ventilation route blocked but water penetrating the outer leaf is transferred directly to the inner leaf via the grout fill, which results in moisture penetration into the building. This moisture can directly cause the decay of timber components built into the structure, as has been seen in an early URM building at one of three schools in Auckland (Auckland Girls Grammar School) which in the early 1990s had their cavities filled with a cementitious grout. As a consequence, dry rot developed in timbers such as door and window frames and skirtings, causing extensive damage. While a filled cavity may seem to be an excellent strengthening solution, it is the ventilation and drainage functionality of a cavity that is the overriding priority.

The filling of a cavity with cementitious grout does not take into account the incompatibility between rigid cementitious mortars and grouts, and the weaker lime mortars that historic (mainly 19th Century and early 20th Century) buildings are constructed of. These materials are incompatible in terms of both strength and permeability, with the difference in permeability potentially leading to a number of detrimental effects on the original performance of the building fabric. The softer, permeable materials, such as bricks and the lime bedding mortar, will become prematurely sacrificial in the weathering process, as the cementitious materials trap water against the more porous, softer elements. As a result, extensive erosion of soft brickwork leads to the loss of original fabric due to the need for brick replacement, as occurred at Auckland Girls Grammar School.

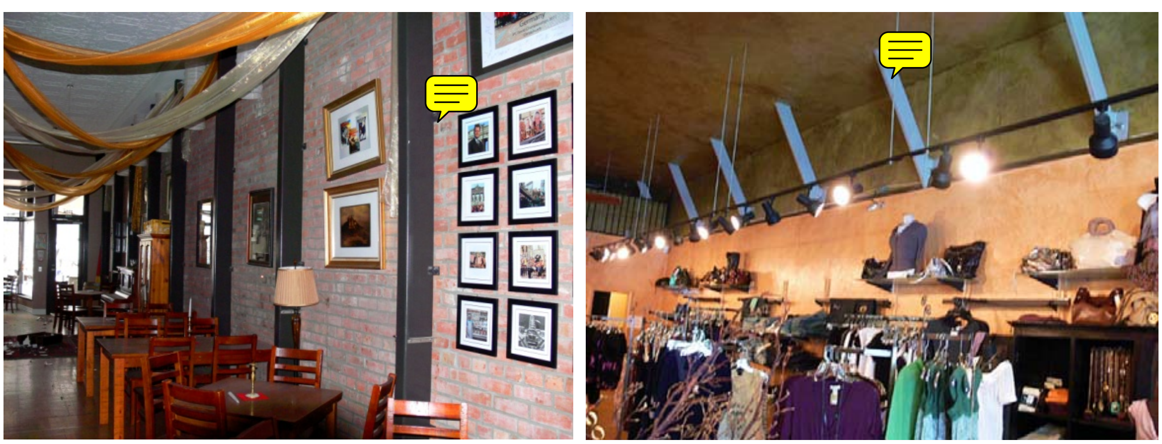
Efflorescence can also develop in structures as a consequence of changing the way that moisture is transferred through a building, and by introducing cementitous grouts and mortars containing soluble salts. This efflorescence can cause extensive damage to both external brickwork and internal plaster finishes.

The current preferred approach to re-attaching the outer leaf is to use a series of proprietary corrosion resistant ties at regular centres which are drilled through the face layer and are epoxy anchored into the structure behind, as shown in Figure 4.6. This technique is effectively a retrofit of the steel ties which have either deteriorated or were omitted in the original construction. The visual impact of these ties is minimal, although care needs to be taken when concealing drilled holes. These ties are irreversible, but their presence is visually negligible.

Figure 4.6 Use of drilled ties to fix external leaf to internal leaf

Inter-Floor Wall Supports

A series of vertical steel sections can be bolted to the inside face of the wall at sufficient spacing to ensure that the width of wall between supports is capable of resisting the out-of-plane forces (see Figure 4.7(a)). These sections act in bending to transfer wall loads to the adjacent floor diaphragms, essentially breaking up a large planar wall into a number of buttressed segments. This simple method may be appropriate in, for example, an industrial building, where visible steel bolted to the walls is in keeping with the character of the building, or in other buildings where the steel can be made to be architecturally appealing. In some other situations it may be less appropriate but less intrusive than other techniques. If there is existing internal framing with space behind for these columns, and no historic material is lost during installation, then it is a ~~perfectly~~ acceptable method. Sections generally fix to the historic material with bolts only, which allows a high degree of reversibility.



(a) Internal strong backs to restrain out-of-plane wall failure

(b) Struts from the floor above to improve out-of-plane performance

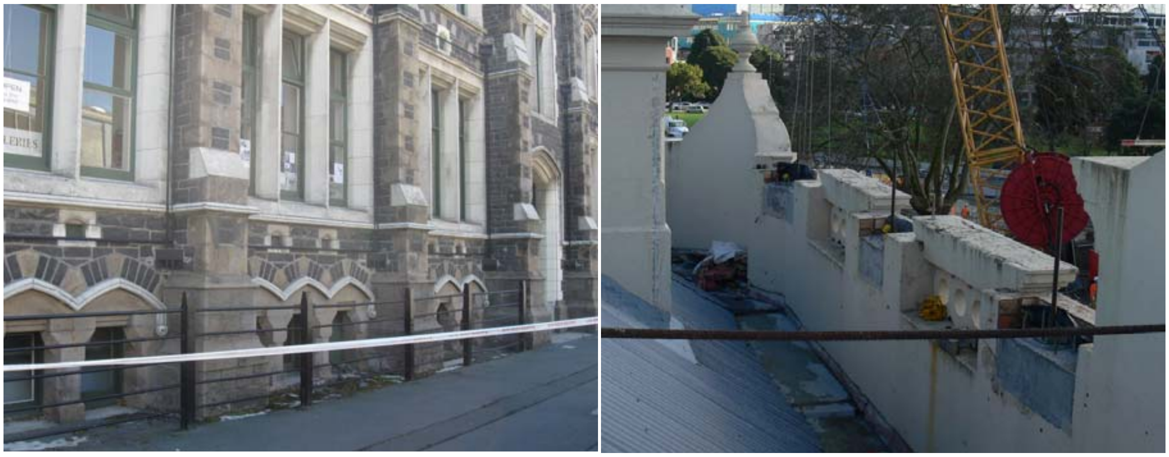
Figure 4.7 Techniques available to increase wall stability against out-of-plane failure

In the past, rather than only supporting the URM walls for out-of-plane actions, these inter-floor wall support systems have been conceived as a method to support the floors in the event that the walls fail and collapse (Cattanach et al., 2008). A technique that is similar to the installation of vertical steel members is to provide a horizontal steel member at the mid-height of the wall and brace this with diagonal struts up to the floor or ceiling diaphragm above, as shown in Figure 4.7(b). This technique might be more suitable than the installation of vertical members if there is a cornice part way up the wall which needs to be conserved, or which can be used to disguise the steelwork. ~~However~~ care needs to be taken to ensure that the struts are visually unobtrusive. Both of these techniques can also be undertaken with the steel substituted with concrete, where this is more appropriate visually, or less commonly with timber. Steel struts can also be recessed within the width of the wall. Recessing the members results in an irrecoverable loss of material and may result in other complications such as cracking, although recesses may be preferable if used beneath a plastered surface, as there it will not affect the interior space. Concrete sections will have larger cross section geometries than will steel sections and will therefore be more intrusive. Also, once cast, concrete is difficult to remove without significant damage, particularly from a porous and naturally coloured material like clay brick. The installation of in-situ concrete is a comparatively permanent measure, so any activity which requires concrete to be cast against brick should be given careful thought before being undertaken.

Post-tensioning

Post-tensioning is an extremely effective method for increasing the out-of-plane strength of URM walls. The post-tensioning may be applied externally as shown in Figure 4.8(a) or be installed internally (see Figure 4.8(b)) by drilling vertical cores through the middle of a URM wall and then inserting steel rods into these cores. The rods may or may not be set in grout, and are then tensioned, which provides an additional compressive force in the wall. This loading modifies the stress behaviour of the URM in bending (i.e. the result of out-of-plane loading). Instead of bending instantly and causing tensile forces, to which URM has little resistance, the wall remains in compression (Ismail et al., 2011). This modification of the material properties also results in an increase in the shear strength of the wall, making post-tensioning an attractive strengthening solution.

Internal post-tensioning has little visual impact, although its installation may be unsuitable in some buildings, as access is required to the top of the wall, and walls need to be of a certain minimum thickness. Drilling cores involves some loss of historic material from the holes, though compared to some methods this is a minor impact. If the bars are fully grouted in place, post-tensioning is essentially irreversible, although this does not necessarily have to be done. The presence of post-tensioning bars is not likely to result in any negative effects to the historic material should their function no longer be required, provided care is taken with all core reinforcement to ensure that it is adequately protected from corrosion. This problem can be completely avoided by using plastic coated steel or FRP bars.



(a) External post-tensioning used in the Christchurch Arts Centre (photo taken after 22 February 2011 earthquake)

(b) Internal post-tensioning bars used in the Birdcage hotel, Auckland

Figure 4.8 Post-tensioned seismic retrofits of URM buildings

There are other methods of core reinforcement, with the most common being non-stressed steel bars set in grout, where the steel reinforcement only becomes stressed when the wall is loaded laterally. The visual impact and reversibility of these methods are the same as for fully grouted post-tensioning, although they are less effective structurally.

Wall reinforcement (FRP and other materials)

There are a number of other methods that may be used to provide out-of-plane stability of unreinforced masonry walls, such as the use of strips of fibre reinforced polymer (FRP) fitted into vertical saw cuts in URM (Dizhur et al., 2010; Dizhur et al., 2011). This technique is known as near surface mounting (NSM). NSM is a relatively recent technique which involves epoxying FRP into saw cuts in the surface of the URM ~~and covering the cut with a grout mixed with brick dust (see Figure 4.5(a))~~. This technique would have some visual impact in naked brick, but little if done within an existing grout line, and none if installed in plastered walls being repointed. This technique can be a particularly effective and non-intrusive method of strengthening, although the finishing of this system is noticeable and work needs to be done to conceal the inserts.

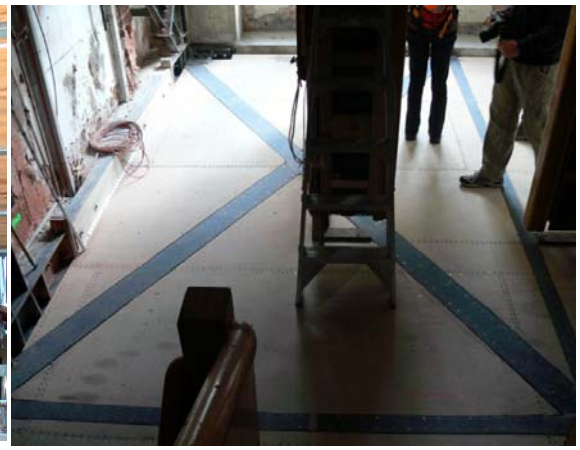
4.2.4 Floor and roof diaphragm stiffening

Aim: Increase ~~in-plane~~ stiffness of horizontal diaphragms (floors and roof) so the seismic forces can be efficiently transferred to masonry shear walls.

Diaphragms are useful because they provide a layer through which lateral forces can be distributed from their source to remote resisting elements, and also act to bind the whole building together at each level. A building which acts as ~~one~~ rigid body rather than a number of flexible panels is far more likely to survive ~~an~~ earthquake. Tying floors to the outer walls (see Figure 4.9(a)) is generally required regardless to ensure that joists are not dislodged (Robinson & Bowman, 2000).

Timber floor diaphragms consist of three main elements; chords, sheathing material, and supplementary structure. To form a diaphragm in a typical URM building, chords need to be established, and mechanical fastenings added to take shear and tensile loads (Rutherford & Chekene, 1990). Several secondary fastenings between the chord and the floor or roof may also be required depending on the technique used. Some tensile ties will penetrate to the outside of the building and others will be drilled and epoxied in place. Existing historic sheathing may prove inadequate and require strengthening or an additional layer of more rigid material (see Figure 4.9(b)).

(a) Steel sections added to stiffen and secure the floor diaphragm



(b) Steel strapping for floor stiffening

Figure 4.9 Examples of floor diaphragm stiffening

Ties to the outside of walls may require metal load spreaders which visually impact the exterior. Many New Zealand buildings display these, and they seem to have become somewhat accepted as part of the strengthening process. Nevertheless, care needs to be taken when considering their visual impact and invisible solutions may be preferable. Much of the additional required work can be hidden within the floor space, but if this is exposed or the connections are extensive, special attention will be required to preserve the visual character of the inter-floor space.

Diaphragm strengthening may have some visual impact if new sheathing material is required. Historic flooring material is often a significant contributor to the character of a place and ought to be retained in view whenever possible. If the existing sheathing is inadequate, a ceiling diaphragm below, or stiffening the existing material might be preferable to covering it. Another approach is to remove the existing sheathing and install a structural layer beneath it. This exercise requires extreme care; firstly because existing sheathing, particularly tongue and groove, is very easily damaged during removal; and secondly, care needs to be taken to restore the boards in the correct order.

Diaphragms which are formed using mechanical connections have a high degree of reversibility; where ties are epoxied into walls there is less reversibility, but minimal visual intrusion. Additional sheathing may damage or alter the nature of the historic timber below, making it less desirable as a solution, although this can be mitigated. Occasionally, pouring concrete over an existing timber floor is considered. This solution can greatly increase the stiffness of the building, but in turn increases its weight and therefore the forces acting upon it. Further, it completely changes the material of the floor and is not a reversible action, because even if it can be removed, the concrete would essentially destroy the character of the underlying timber. This procedure is therefore not recommended except in ~~exceptional~~ circumstances.

Roof diaphragms where the structure is exposed are slightly different, as the inclusion of a plywood diaphragm *above* timber sarking is generally acceptable if this area can be accessed, for example if the roofing is being replaced. This installation can also help to protect the sarking beneath. Roofs with suspended ceilings can be made to accommodate cross bracing, struts, and more innovative solutions, as they can be hidden within the ceiling space. In instances where the roof provides little diaphragm action, or the forming of a diaphragm is uneconomical or impossible, a horizontal load resisting member at the level of the top of the walls can be used to provide stability to the walls under out-of-plane loads. However, this member needs to be fixed to stiff elements at regular intervals to transfer horizontal loads, and these stiff elements may need to be introduced to the building if other structure cannot perform this task.

4.2.5 **Connection of structural elements**

Aim: ensure adequate strength of roof-to-wall, floor-to-wall and wall-to-wall connections. Good connectivity between the walls and the floor and roof diaphragms will ensure that the walls only deflect outwardly over the height of one storey of a building. This reduces the out-of-plane displacements that lead to wall collapse. Similarly, good connectivity along the vertical intersection of walls meeting at corners of a building (or internal walls meeting with an external wall) will ensure that the building responds as a single structural system and not as separate, isolated components. Much better performance can be expected in an earthquake when the building responds as a single system.

The most problematic deficiency in URM construction is inadequate connection of diaphragms to walls (FEMA, 2006), as failure of these connections can potentially lead to global collapse of the building. The addition of a network of small ties can substantially increase the strength of the building by fixing the walls to the floor and roof diaphragms (Robinson & Bowman, 2000). These ties need to resist two actions: shear from the diaphragms trying to slide across the walls; and tension from the diaphragm and wall trying to separate. If these ties are missing, the walls will be acting as a cantilever from the ground level under lateral loads, and floors and roofs are far more likely to be dislodged from their supports, which is the most common mode of failure for URM buildings in an earthquake. This failure mode is shown in Figure 4.10.

Figure 4.10 An extreme case in the 2010 Darfield earthquake where inadequate connections have resulted in wall collapse (Welstead House, 184-188 Manchester Street)

The use of simple metal anchors to connect the walls to the floor and roof diaphragms is relatively straight forward and was observed in many buildings that survived both earthquakes (see Figure 3.14 and Figure 3.15). Recently, some proprietary systems have become available that use steel reinforcement to connect walls to the floor and roof diaphragms, and to provide wall-to-wall connection at corners and other wall intersections. Typically, the reinforcement is placed in horizontally cored holes that pass through the entire building at each floor level and at the roof level. The reinforcement is then post-tensioned and grouted in order to clamp the walls to the floors and roof and to each other. In some applications, vertical reinforcement, sometimes with post-tensioning, is also used to increase the compressive stress in the wall which results in an improvement to the walls earthquake strength when subjected to horizontal loads.

4.2.6 **Shear walls**

Aim: Provide additional storey/base shear strength; this could be through strengthening existing walls or by construction of additional shear walls.

Most URM walls are required to transfer some degree of shear loading along their length. If a building has insufficient shear capacity in a particular direction, then capacity of existing walls can be increased instead of inserting additional structure. There are various methods for achieving this strength increase which generally involve the application of an additional layer of material bonded to the surface of URM to increase its strength, although there are some measures which involve altering the wall itself, such as post-tensioning, as described above. Most of these measures involve a plane of extra ~~independent~~ structure being applied over the surface of the URM, effectively forming new shear walls, which are described below.

The presence of openings in a shear wall renders ~~that section~~ less stiff than the surrounding full height walls, meaning that the wall above and below, or between closely spaced openings, will likely be the first areas to fail in the event of an earthquake. Infilling the openings will eliminate this problem by making the wall continuous, and has been advocated as a valid solution in the past. Problems with altering the character of the building and matching brick and mortar colours mean that this approach should only be used as a last resort and even then preferably not in visible areas. Infilling openings is likely to be somewhat reversible if done with brick, but not completely, and visual impact will depend on the location. If in-filled with concrete, the work will be less reversible and the ductile behaviour of the wall may be affected due to incompatible stiffnesses. Localised steel cross bracing near openings is another technique which can prove effective, but again this system is likely to be highly visible and should only be undertaken when it does not detract from the character of the building.

Shear walls are used to increase the strength of existing URM walls or are added as new elements. Materials which resist shear loads can be added to the surface of the URM; these might include gypsum plasterboard, particle board, plywood, or plate steel (Robinson & Bowman, 2000), and are generally fixed to the URM wall with bolts via a supplementary structure. This approach leads to the surface of the URM wall generally being covered which may interfere with decorative elements on walls and openings, although this interference can be alleviated by using stronger materials such as plate or strap steel. They can also increase the thickness of the wall, which is not particularly desirable as it can reduce the scale and area of the interior. For these reasons shear walls can be visually detrimental if used indiscriminately. Stand alone shear walls, which are independent of URM walls, can be introduced, although these can be detrimental for similar reasons. Despite these negatives, shear walls are a practical and efficient method for strengthening and are commonly used. All of these materials can be easily removed in the future, which makes them good solutions for shear walls in two to three storey buildings with moderate horizontal loads.

The shotcreting of shear walls was a common strengthening technique during the 1980s. This technique involves spraying concrete onto the surface of a URM wall to essentially cast a new wall against the existing wall, as shown in Figure 4.11(a). This technique provides plenty of additional strength to the wall, both in-plane and out-of-plane, but is now largely regarded as unacceptable unless absolutely necessary. The technique causes a significant increase in wall thickness and it is very difficult to remove the concrete, and even more so to restore the wall behind to any semblance of its character prior to concreting. Furthermore, the installation of shotcrete generally requires the building to be gutted, which results in the loss of much heritage material and creates an essentially new interior (Robinson & Bowman, 2000).

(a) Shotcrete applied to a former URM building **(b) Surface bonded FRP applied to the exterior of a URM wall**

Figure 4.11 Examples of strengthened masonry shear walls

Another technique for forming strengthened shear walls is the addition of surface bonded fibre reinforced polymer sheets is in Figure 4.11(b). These sheets do not require the same invasive installation as shotcrete walls, but generally are equally permanent, and have potentially limited application, although new technology may soon change this. If it is possible to provide out-of-plane strength using FRP inserts, coupled with an FRP surface layer for shear, then this solution could be far superior to shotcrete from an architectural perspective. An important consideration with the use of sheets of FRP is that it is impermeable, which can lead to problems with water trapped within the building resulting in damp and mould issues, and potential de-bonding of the epoxy.

(a) Exterior view of retrofitted building **(b) Close up view of surface application**

Figure 4.12 Textile reinforced render as retrofit

4.2.7 Insertion of internal frames

Aim: Provide alternative structural system to resist the seismic loads.

Moment frames

Moment frames are a common method of gaining additional horizontal resistance which can also be used as a local strengthening solution. The advantage of this system is that it is comprised of beams and columns, so is fully customisable, and there is space between the vertical and horizontal elements. Moment frames allow full visual and physical access between each side of the frame, and minimal spatial disruption. In building façades with numerous openings, some form of moment frame can often be fitted to the masonry piers on the inside or outside (or both) depending on the effect on the architectural character. Moment frames can be a particularly effective solution, especially where the frame is tailored to the character of the building. Care needs to be taken with steel frames in particular to ensure stiffness compatibility with the existing structure (Robinson & Bowman, 2000). Steel is a ductile material, but URM is not, meaning that under earthquake loads the added stiffness of the steel might not come into effect until a load is reached where the URM has already been extensively cracked.

Moment frames can be an excellent strengthening technique, either to supplement an existing wall or as a new, stand alone element. If a steel frame is erected against an existing wall where weakness exists, the frame needs to be fixed either directly to the URM using bolted connections into the wall or to the diaphragm (see Figure 4.13(a)). Installing concrete frames is a more complex undertaking, as these will often be constructed by thickening existing piers, although a concrete frame which is separate from the existing structure is possible (see Figure 4.13(b)). In both situations it is important that architectural character is retained, and historic material conserved. Some considerate and artful design strategies may need to be undertaken to achieve this.

Steel moment frames have a high degree of reversibility, as again they rely on mechanical connections and relatively small ties to connect to the existing structure. Concrete frames are generally far less reversible, but can sometimes be better concealed when this is a requirement. Figure 4.13(b) shows a large new moment frame which is expressed as a new element. Some recent buildings have very effectively used precast concrete load-resisting elements which are separate from the URM walls, solving the problem of reversibility (Cattanach et al., 2008).

(a) Post-earthquake condition of a URM building having an internal steel frame retrofit

(b) Reinforced concrete moment frame retrofit

Figure 4.13 Internal moment frames installed as seismic retrofits

Braced frames

Braced frames are available in various configurations: concentric, tension only concentric, eccentric, and 'K' bracing. The key functional difference between braced frames and moment frames is that due to the diagonal braces, braced frames prevent physical continuity between spaces on either side of the frame. Braced frames are also generally constructed from steel rather than concrete, and are much more rigid than moment frames.

Braced frames are a very efficient method of transferring horizontal forces but have significant setbacks. Their use in façade walls is usually precluded by the presence of windows, as diagonal braces crossing window openings are generally considered to be poor design. It is also difficult to get a braced frame to conform to an existing architectural character; however they can be used to very good effect within secondary spaces, and can be made to fit architecturally in some situations with careful consideration. Figure 4.14 shows braced frames in use. Generally speaking, steel braced frames have a good degree of reversibility and can provide excellent strengthening when used appropriately.

(a) Eccentric bracing in a walkway

(b) Eccentrically braced core

Figure 4.14 Eccentrically braced steel frame retrofits (photos courtesy of Dunning Thornton Consultants)

4.2.8 Removal of mass and/or geometric/stiffness irregularities

Aim: Reduce the seismic forces through reduction of structural mass or structural irregularities.

Another approach to seismic improvement of URM buildings derives from its weight. Seismic actions are directly proportional to the mass of the building, so if mass is reduced, so are the forces acting upon the building. A lighter building requires less lateral strength and therefore less additional strengthening. Reducing the mass of a building may seem at face value to be a sensible approach; however past experience has shown this to not be so. The mass must be removed from somewhere, and the higher up the mass is, the stronger the forces upon it and the more difficult it is to strengthen, so the top of the building is the first place which has been looked at. Historically this logic has led to the ad-hoc removal of decorative elements such as parapets, gables, chimneys, and occasionally whole towers (Robinson & Bowman, 2000). These elements will almost always significantly contribute to heritage value and character, and their retention is essential to preserving these attributes. Indeed, it is often desirable to replace these features if they have been removed from buildings and still exist. While reduction of weight may be achieved in more minor ways, such as removal of internal URM partitions or the removal of plant loads, the wholesale removal of decorative elements is strongly discouraged.

Section 5:

Set of representative buildings

In this section a recommendation is made for a set of unreinforced masonry (URM) buildings that are representative of the Christchurch URM building stock in terms of both their architectural characteristics and their observed earthquake performance. This section was prepared in response to the scope of the report as requested by the Royal Commission and outlined in section 1.1. Several iconic stone masonry buildings are first identified, recognising their historic significance to the people of Christchurch and their contribution to the character of the city. A selection of clay brick buildings is then presented for consideration, with attention first given to the performance of clay brick building that had been retrofitted, followed by details of several unretrofitted building that currently remain, and concluding with a selection of clay brick buildings that have since been demolished.

For each building a short description of the character and history of the building is provided, followed by a brief explanation for the reason why this building is recommended for consideration by the members of the Royal Commission as being representative of URM construction throughout New Zealand.

5.1 Stone masonry buildings

5.1.1 Christchurch Cathedral

The cornerstone of Christchurch Cathedral was laid on 16 December 1864, but financial problems saw the Cathedral's completion delayed between 1865 and 1873. In 1873 a new resident architect, New Zealander Benjamin Mountfort, took over the project and construction began again. The nave and tower were consecrated on 1 November 1881, but other parts of the Cathedral were not finished until 1904. The Cathedral underwent major renovations during 2006–2007, including the replacement of the original slate roof tiles. The February 2011 Christchurch earthquake destroyed the spire and part of the tower – and severely damaged the structure of the remaining building. The Cathedral had been damaged previously by earthquakes in 1881, 1888, 1901 and 2010[9].

Christchurch Cathedral occupies a position of prominence at the centre of the Christchurch Square which is in the centre of the CBD. For many people the damage to the Cathedral has been a defining image of the events in Christchurch since 4 September 2010. The Cathedral's masonry construction is complex, with dressed outer stone and a clay brick interior. Anchor plates that were installed in the gable end wall above the rose window (see Figure 5.1(a)) helped to secure the wall during the 22 February 2011 earthquake, presumably enabling those within the Cathedral at the time to safely exit through the front door. Unfortunately the rose window sustained damage on 13 June 2011. The building is recommended for attention largely for its historic significance, but also because it is currently anticipated that the Cathedral will be rebuilt. Structural improvements to the Cathedral prior to 4 September 2010 appear to have been effective, but clearly have not prevented substantial damage to the building. See Figure 3.30 and Figure 3.34 for further images of the Cathedral.

(a) Condition after 22 February 2011 **(b) Condition after 13 June 2011**

Figure 5.1 Damage to Christchurch Cathedral

5.1.2 Christchurch Basilica

The Cathedral of the Blessed Sacrament, commonly known as the Christchurch Basilica, was designed by architect Francis Petre. Construction started in 1901 and was complete by 1905. The Basilica was designed in the neo-classical style and is faced in Oamaru limestone. The solid walls are constructed of reinforced concrete and faced in stone. The roofs to both bell towers and the east dome are timber framed with a copper finish. The

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[9] This text is taken from: http://www.historic.org.nz/TheRegister/RegisterSearch/RegisterResults.aspx?RID=46

nave roof is timber framed and finished in terracotta tiles. The flat roofs east of the nave around the base of the dome are constructed of reinforced concrete. The building is held to be the finest renaissance style building in New Zealand and the most outstanding of all Petre's many designs[10].

The Basilica is a complex structure exhibiting characteristics of both unreinforced masonry and early concrete construction (see Figure 3.27(b)). The primary reason for identifying this building for attention is because of its distinctive architectural character as it is not particularly representative of a larger stock of buildings in New Zealand. Currently the principal concern regarding the stability of the Basilica is to deconstruct the dome because of the falling hazard posed by the damaged drum at the dome base (see Figure 5.2(a)). Damage to the Basilica's clock towers (see Figure 5.2(b)) suggests parallels with the collapse to the spire of the Christchurch Cathedral as shown in Figure 5.1.

(a) Damage to drum at base of dome **(b) Damage to clock towers**

Figure 5.2 Damage to the Christchurch Basilica (images taken post-February 2011)

5.1.3 Canterbury Provincial Council Buildings

The foundation stone for the Canterbury Provincial Council Buildings was laid in January 1858. The first set of buildings were a two-storey timber building, forming an L shape along the Durham Street frontage, with the Timber Chamber, modelled on 14th and 16th century English manorial halls, being the meeting room for the Provincial Council. The Stone Chamber was the new meeting room for the council; it was larger than the Timber Chamber to cope with an increased size of the council. The Stone

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[10] This text is reproduced with modifications from: http://www.historic.org.nz/TheRegister/RegisterSearch/RegisterResults.aspx?RID=47

Chamber's interior was described as provincial architect Benjamin Mountfort's most impressive achievement[11].

This set of buildings is recommended for further attention both because of the historic significance of the buildings and because the failure mode observed for the stone masonry construction (see Figure 5.3) is representative of failures observed in other stone masonry buildings, and in particular several stone masonry churches. See also Figure 3.35.

(a) Stone masonry collapse

(b) Collapse of the Stone Chamber

Figure 5.3 Earthquake damage to the Canterbury Provincial Council Building (images taken post-February 2011)

5.1.4 Christchurch Arts Centre

> "The Arts Centre in Christchurch is a collection of fine Gothic Revival buildings, formerly used by the Canterbury University College (now the University of Canterbury) and two of the city's secondary schools. Construction on the buildings for the Canterbury University College, which later became the University of Canterbury, began with the building of the Clock Tower block. This building, which opened in 1877 and was designed by Benjamin ~~Woolfield~~ Mountfort, was the first building in New Zealand to be designed specifically for a university"[12].

The Christchurch Arts Centre complex is composed of stone masonry buildings that merit investigation because of the number of seismic retrofit technologies that have been previously installed within the complex. Three technologies in particular merit attention, being the innovative use of horizontal and vertical unbonded post-tensioning that appears from the exterior to have been highly successful in preventing damage (see Figure 5.4(a)), the use of wall-diaphragm anchor plates that in most cases have

[11] This text is reproduced with modifications from: http://www.historic.org.nz/TheRegister/RegisterSearch/RegisterResults.aspx?RID=45

[12] This text is taken from: http://www.historic.org.nz/TheRegister/RegisterSearch/RegisterResults.aspx?RID=7301

effectively restrained the major part of gable end walls although some damage at the top of gables has occurred (see Figure 5.4(b)), and the use of surface bonded fibre reinforced polymers to the interior of the building. Documenting the successful performance (or otherwise) of these technologies will be useful when considering appropriate seismic improvement techniques for other iconic stone masonry buildings. See also Figure 3.32, Figure 3.38 and Figure 3.40.

(a) Good performance of stone masonry building with horizontal and vertical external post-tensioning

(b) Poor performance of stone masonry tower and top of gable

Figure 5.4 Mixed performance of the Christchurch Arts Centre (images taken post-February 2011)

5.1.5 Former City Malthouse

The Malthouse is a stone masonry building that was constructed in 1867-1872 (see Figure 5.5). The Malthouse is one of New Zealand's oldest buildings[13] has three levels, with a half basement, timber floor and roof diaphragms and an irregular floor plan. The building was used as a Malthouse until 1955, when it was converted to the Canterbury Children's Theatre. Between 1972 and 1984 the Malthouse went through several architectural renovations that included seismic retrofit. The roof was raised in two stages: the first stage involved raising half of the roof in 1992 and the second stage involving raising the remainder of the roof in 2003. Seismic retrofit of the Malthouse in 2003 was found to be insufficient and consequently the building's lateral load resisting system was again updated in 2008. The seismic retrofit involved injecting grout into the rubble masonry walls, strengthening the roof by introducing new steel trusses (see Figure 5.5(b)), strengthening of the floor diaphragms by replacing the plywood and introducing additional timber blocking (see Figure 5.5(c)), and installing new wall-diaphragm anchors. It was established from discussions with the manager that the cost of retrofit was approximately $NZ 750,000. The building appears to have performed well.

[13] http://www.historic.org.nz/TheRegister/RegisterSearch/RegisterResults.aspx?RID=1902

(a) Exterior view

(b) Interior view

(c) Additional blocking at ground floor

(d) Wall-floor diaphragm connection

Figure 5.5 Former City Malthouse (images taken post-September 2010)

5.2 Retrofitted clay brick masonry buildings

In general, retrofitted URM buildings performed well in the 4 September 2010 earthquake, with minor or no earthquake damage observed. Partial or complete collapse of parapets and chimneys were amongst the most prevalent damage observed in retrofitted URM buildings, and was attributed to insufficient lateral support of these building components. Out-of-plane separation of the façade from the side walls was observed in some URM buildings, and was the result of insufficient wall-diaphragm anchorage. Most of these seismic retrofits were more severely tested in the 22 February 2011, with mixed success.

Most of the retrofitted URM buildings had significant heritage value based on their era of construction and aesthetic quality and therefore a carefully considered, minimally invasive retrofit solution had been preferred. The addition of a secondary structural system was found to be a common retrofit solution, with fewer buildings adopting alternative solutions such as steel strapping, the application of surface bonded fibre reinforced polymer (FRP) sheets, and post-tensioning. Case study examples of the performance of retrofitted URM buildings that were inspected following the 4 September 2010 earthquake are briefly reported below.

5.2.1 The Smokehouse, 650 Ferry Road

The Smokehouse, located at 650 Ferry Road, is a two storey isolated clay brick URM building as shown in Figure 5.6. The building's construction date can be confirmed as pre-1930's, and the building has been categorised as a heritage building by the Christchurch City Council. The building's foot print is approximately square, having dimensions of 13 m along Ferry Road and 10 m along Catherine Street. The original mortar is a weak lime/cement mortar with large grain size sand. In places the original mortar was re-pointed with strong cement mortar.

(a) Exterior view **(b) Interior view showing added steel frames**

Figure 5.6 The Smokehouse, 650 Ferry Road (images taken post-September 2010)

The Smokehouse was seismically retrofitted in 2007 by introducing secondary moment resisting steel frames. This retrofit also included alterations to the internal layout, which involved partial removal of original external walls and replacement with moment resisting steel frames that created openings into the adjoining new section of the building (see Figure 5.6(b)), and also infilling one window at the second floor level. The retrofit design of the building won the New Zealand Architectural Award in 2008 for initiative in retention, restoration and extension of a significant building and its adaption to new uses[14]. The building appears to have performed well, with no significant signs of earthquake damage observed.

5.2.2 TSB Bank Building, 130 Hereford Street

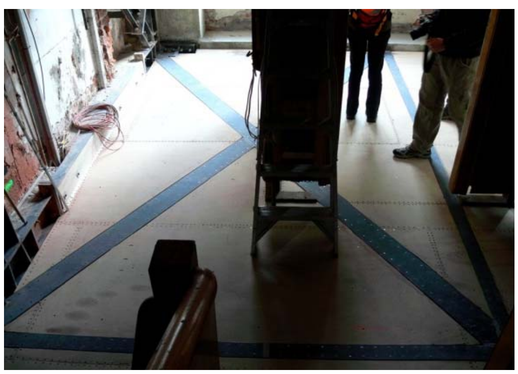
130 Hereford Street is a 1920's 3+ storey isolated URM building, currently owned and occupied by TSB Bank Limited. The original structural system consisted of URM load bearing clay brick walls, built in the Chicago style architecture as shown in Figure

[14] Smokehouse Restaurant. *Smokehouse*. 2009. Retrieved 28 October 2010. Available from: http://www.holysmoke.co.nz/.

5.7(a). The estimated footprint area of the building was approximately 450 m². The ground floor has been re-furbished and is occupied by the TSB Bank, whereas the upper levels required refurbishment at the time of the 4 September 2010 earthquake and therefore the retrofit structure was exposed at the time of inspection. A weak lime mortar (scratched with a finger nail) was used in construction. The bricks used were bright red burnt clay bricks, laid in a common bond pattern. The building has flexible timber diaphragms that consist of plywood sheathing resting over timber joists that are supported on the load bearing URM walls.

(a) Exterior view

(b) steel brace frame



(c) floor diaphragm strengthening

(d) gable strengthening

(e) roof diaphragm strengthening

Figure 5.7 130 Hereford Street (images taken post-September 2010)

The building was seismically retrofitted by the new owner (TSB Bank) in 2009, which involved the introduction of secondary frames. The facade is strengthened by concrete columns and beams at the floor levels (forming a concrete frame) and the side walls are strengthened using steel frames with diagonal braces that are anchored into the masonry as shown in Figure 5.8 and Figure 5.7(b). The floor diaphragms on levels 2 and 3 were stiffened with plywood sheets and 'X' pattern steel plates, with screw fixings spaced at approximately 20 mm to connect the plates to the timber diaphragm as shown in Figure 5.7(c). Figure 5.7(d) shows strengthening of the gable, consisting of steel frames secured with adhesive anchor bolts, and Figure 5.7(e) shows the roof diaphragm strengthening using steel tie rods. The walls are supported by newly added concrete beams at the basement level, further resting over old concrete basement walls.

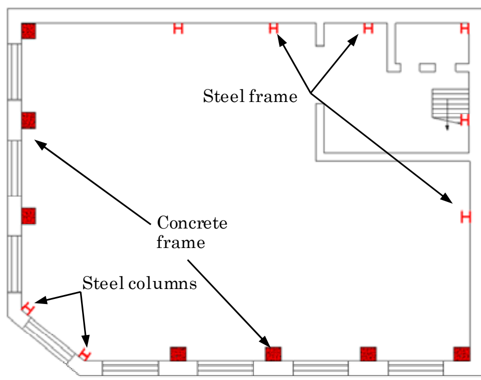


Figure 5.8 Floor plan of TSB Bank Building, showing retrofit

5.2.3 X Base Backpackers, 56 Cathedral Square

This four storey URM building located in the northeast corner of Cathedral Square was constructed in 1902 (see Figure 5.9(a)). The building, formerly known as the Lyttelton Times building and now occupied by X Base Backpackers, is the last in a row of multi-storey buildings on Gloucester Street and butts up to the original Canterbury Press building. The building's exterior aesthetics are similar to the nineteenth century Chicago high-rise buildings (i.e., Romanesque style), with heavy vertical URM piers ending in round headed arches on the front façade and two leaf thick solid brick URM walls on the periphery. ~~The facade of the building is shown in Figure 5.9(a)~~. The building was registered as a category I heritage building with the New Zealand Historic Places Trust in 1997 and therefore an application for its demolition was declined and the building was instead purchased by the Christchurch Heritage Trust. The building was constructed using bright red burnt clay bricks, laid in a common bond pattern. From preliminary scratch tests it was established that a weak lime/cement mortar was used in construction, with variation in the mortar strength in upper floors.

(a) Exterior view

(b) 'X' steel brace fixed into wall

(c) Interior steel frames

(d) Parapet restraints

(e) Façade separation

(f) Cracking through the spandrel on ground level

Figure 5.9 X Base Backpackers, 56 Cathedral Square (images taken post-September 2010)

The X Base Backpackers building was seismically retrofitted in 2001 using steel moment resisting frames. The moment frames, as seen from the fourth floor of the building, are shown in Figure 5.9(c). As part of the seismic retrofit scheme, the parapets were tied back to the roof structure using hollow steel circular sections (see Figure 5.9(d)). In a recent inspection of the building, steel straps anchored to the side walls in an X pattern were also observed (shown in Figure 5.9(b)) and were possibly part of an earlier retrofit scheme.

5.2.4 Vast Furniture / Freedom Interiors, 242 Moorhouse Avenue

Vast Furniture / Freedom Interiors (shown in Figure 5.10) is a single storey masonry building, with its roof supported on strong steel trusses that were laterally braced by connecting steel section trusses. Masonry materials observed were bright red clay bricks and a weak lime mortar, with URM laid in a common bond pattern.

(a) wall-diaphragm anchor punching

(b) wall strengthening using steel sections

(c) interior of the building (location where anchor plate pull out occurred)

Figure 5.10 242 Moorhouse Avenue (images taken post-September 2010)

The trusses are further supported on steel portal frames, but the frames had more modern welded joints than the old fashioned riveted joints used in trusses, which suggests that the portal frames were added later to the building as a seismic retrofit solution (see Figure 5.10(b) and (c)).

During the 4 September 2010 earthquake the parapets collapsed out-of-plane and the wall-diaphragm anchors pulled out from the wall, with the anchors punching through the brickwork and creating localized wall damage (refer to Figure 5.10(a)). The building was cordoned off as falling hazards had been identified during post-earthquake evaluation but the internal retail area remained open.

5.2.5 **Environment Court Ministry of Justice, 282-286 Durham Street North**

The Environment Court building is a one storey isolated URM building that was constructed in the 1890's. The building was originally constructed as an art gallery, with street facades divided into a series of bays and decorated with patterned cornices. A wooden truss supports a gable roof and rests on load bearing URM walls. Due to the building's historic value it is identified as a Category I historic place on the New Zealand Historic Places Trust register.

(a) Armagh street view **(b) Strap detail around openings**

Figure 5.11 282-286 Durham Street North (images taken post-September 2010)

The building was seismically retrofitted in 1972 by the Justice Department. The seismic retrofitting scheme involved the addition of cross walls and strapping of the building with steel plates, as shown in Figure 5.11.

5.2.6 Shirley Community Centre, 10 Shirley Road

Shirley Community Centre is a single storey URM building that was constructed in 1915 to be the Shirley Primary School. The building has a hipped roof and was constructed in the Georgian style with large and regular fenestrations. This historic building was registered under the Historic Places Act in 1993. The perimeter cavity walls consist of two leaf thick solid red clay brick masonry with a single veneer yellow brick layer on the exterior surface.

(a) FRP sheet strengthening, image taken 2009 (BBR Contech, 2010) **(b) FRP strengthening after 4 September 2010 earthquake, no visible cracking**

Figure 5.12 Shirley Community Centre

Seven individual wall areas were strengthened with surface bonded FRP sheets using Sikawrap 100G (the application of FRP retrofit is shown in Figure 5.12(a)) in the locations shown in Figure 5.13. FRP rod anchors were installed to bond the applied

Sikawrap 100G sheet to the concrete foundation beams (BBR Contech, 2010). The out-of-plane stability to the perimeter wall was provided by using steel hollow sections as strong backs fixed to the URM walls. To ensure sufficient lateral load resistance in the North-South direction a concrete shear wall was also added at the location shown in Figure 5.13. The veneer brick layer was secured to the main wall using helical veneer ties at regular spacing.

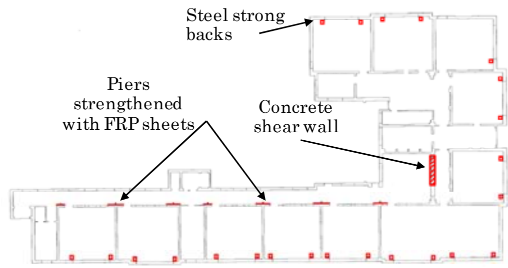


Figure 5.13 Floor plan of Shirley Community centre showing retrofit

5.2.7 Review of performance of retrofitted clay brick URM buildings

The above details were documented following the 4 September 2010 earthquake, where most retrofitted clay brick buildings performed well. The subsequent performance of these buildings is briefly summarised in Table 5.1.

Table 5.1 Performance of retrofitted clay brick URM buildings

Building and address	Assessed earthquake performance
The Smokehouse, 650 Ferry Road	**All:** No significant damage. See Figure 5.6.
TSB Bank, 130 Hereford Street	**September:** Some cracks in the basement walls. Retrofit appeared to perform well. See Figure 5.7. **December:** Unknown. **February:** ~~Gable~~ failure on the ~~east~~ side, but again the retrofit appeared to have performed well. **June:** No further significant damage.
X Base Backpackers, 56 Cathedral Square	**September:** Some cracking at top of front facade. Timber shoring placed at top of parapet (visible in Figure 5.9(a)). **December:** Unknown. **February:** Front facade was in process of being repaired, and was covered in scaffolding. Observed damage includes failure of the north ~~east~~ corner at top floor (rear of the building), extensive cracking of front facade (particularly in spandrels). Parapet strengthening appeared to work well, apart from where walls failed. Diagonal shear cracking and failure of some walls of top storey rooms, not visible from the street. X steel straps appear to have kept the walls from collapsing. **June:** Unknown. This building has recently been demolished.
Vast Furniture, 242 Moorhouse Ave	**September:** Partial punching shear of wall-diaphragm anchor, parapet collapse. See Figure 5.10.

	December: Unknown. **February:** Wall-diaphragm anchors punched through further, but no collapse. **June:** No further significant damage externally visible.
Environmental Court, 282-286 Durham Street North	**September:** No apparent damage. See Figure 5.11. **December:** Unknown. **February:** Although the retrofit behaved well the building has suffered some damage, particularly around the entrance. **June:** No further damage.
Shirley Community Centre, 10 Shirley Rd	**September:** No visible damage. See Figure 5.12. **December:** Unknown. **February:** Differential movement between cavity wall layers causing veneer ties to become visible. Liquefaction and differential movement around the grounds. Some cracks extended from ground into the building. Movement of the roof diaphragm visible. In-plane cracking of external walls. **June:** Out-of-plane collapse of external veneer layer.

5.3 Unretrofitted clay brick buildings

5.3.1 127-139 Manchester Street

127-139 Manchester Street is a 3 storey clay brick URM "L" shaped row building that was originally constructed circa 1905 and is listed by the Christchurch City Council as a protected building[15]. The building consists of 7 'bays' along Manchester Street, each having an approximate length of 5 m, with an overall building height of approximately 12 m as shown in Figure 5.14.

(a) 135-139 Manchester Street, out-of-plane facade collapse

(b) 139 Manchester Street, through steel anchors and rotten timber roof diaphragm

Figure 5.14 Damage to clay brick URM building at 135-139 Manchester Street (images taken post-September 2010)

[15] Christchurch City Council. "Protected Buildings, Places and Objects in Christchurch City Coucil". Retrieved 25 October 2010. Available from: http://ketechristchurch.peoplesnetworknz.info/canterbury_earthquake_2010/topics/show/172-list-of-protectedbuildings-places-and-objects-in-christchurch-citycouncil.

The first storey load bearing walls of the building are solid and four leaves thick and the upper storey walls are solid two leaves thick clay brick masonry. The front facade wall of the building is two leaves thick for the upper level and three leaves thick for the first level. All brickwork was constructed in the English bond pattern. Internal non-load bearing partition walls were constructed using timber studs with lath and plaster type finish. The ground floor was modified using a combination of concrete and timber supporting structure in order to provide larger open shop front space. Canopies extended along Manchester Street above the ground level and were tied back into the piers of the first level using steel rods. Decorative, balustrade type parapets extending approximately 1 m above the roof level were positioned around the street frontage perimeter.

The corner bay of the building (139 Manchester Street) was in a deteriorated condition and had been poorly maintained, with visible water damage and rot of the timber floor and roof diaphragms being evident. The floor joists and roof rafters were oriented in the North-South direction for the building portion along Manchester Street. The end gable was connected to the roof structure using only two through anchors with round end plates.

The building sustained considerable damage during the 4 September 2010 earthquake, mainly concentrated at the end bay (139 Manchester Street) where the front facade entirely collapsed out-of-plane (see Figure 5.14). The entire building sustained damage from collapsed parapets, apart from two bays (135 and 137 Manchester Street) where the parapets remained on the building. From visual observations and physical assessment of the collapsed masonry the mortar was found to be in a moist condition and the mortar that was adhered to the bricks readily crumbled when subjected to finger pressure (see Figure 3.3), suggesting that the mortar compression strength was low ~~(<2 MPa)~~. The collapsed facade wall revealed extensive water damage to the timber structure, with rotten floor joists and roof rafters. Also, it was observed that there were large patches of moist masonry on the interior surface of the building, especially around the roof area (there was no precipitation during the period following the earthquake and prior to building inspection).

It appears that the through steel anchors at the gable did not provide sufficient restraint to the masonry, with the brickwork being pulled around the steel anchor plates. Furthermore, from images prior to the earthquake it is evident that there were significant cracks through the spandrel and the parapet over the top corner window of 139 Manchester Street. Falling parapets landed on the canopies, resulting in an overloading of the supporting tension braces that led to canopy collapse. The connections appeared to consist of a long, roughly 25 mm diameter rod, with a rectangular steel plate (approximately 5 mm thick) at the wall end that was approximately 50 mm wide x 450 mm long and fastened to the rod, and was anchored either on the interior surface or ~~within the centre~~ of the masonry pier or wall. The force on the rod exceeded the capacity of the masonry, causing a punching shear failure in the masonry wall (see Figure 5.15).

Figure 5.15 137 Manchester Street, pull out of the canopy supports

5.4 Clay brick URM building that have been partially or fully demolished

5.4.1 192 Madras Street

This building was designed by the Christchurch architectural firm of England Brothers and was constructed in approximately 1918-1919 on a narrow plot on the east side of Madras Street (see Figure 5.16). The building was not listed with the New Zealand Historic Places Trust but had significant historical and social significance as the original headquarters of the Nurse Maude Association. The building was gifted to the Nurse Maude Association and Nurse Maude herself lived in the building's upstairs flat and died in the property in 1935. The building was turned into office space in the mid 1990s (Christchurch City Council, 2010).

(a) cracking through top spandrel **(b) in-plane diagonal cracking through top spandrel**

Figure 5.16 Performance of 192 Madras Street (images taken post-September 2010)

The building had a footprint of approximately 8.8 m by 27 m, with one heavily perforated wall located on the western side (facade) and the other three walls having minimal perforations. The construction was unreinforced masonry with wooden diaphragms and a lightweight roof. The external walls were solid load-bearing masonry and stepped from three leaves to two leaves at the first floor level and to one leaf at parapet level. Diaphragm anchors at the first floor and roof level were installed in 1998, providing some earthquake strengthening, but no remedial strengthening work was applied to the facade wall.

Comprehensive damage was visible to the facade wall following the 4 September 2010 earthquake, with the spandrel panels at the first floor and roof level having extensive cracking, both vertically and diagonally. There appeared to be some movement of the facade at the diaphragm level in the horizontal direction perpendicular to the plane of the wall. The side walls suffered diagonal shear failures that were visible internally, extending into the stairway wells. The parapet remained attached, as it was supported to some extent by masonry columns that were an extension of the side walls. A diagonal crack extended from the intersection between the top east corner of the side wall and the masonry column diagonally down (see Figure 5.16(b)), indicating possible rocking of the parapet block out-of-plane.

5.4.2 Joe's Garage Cafe, 194 Hereford Street

At the time of construction in the 1920's, 194 Hereford Street was the end building in a row of two storey buildings. The building was a two storey isolated URM building most recently occupied by Joe's Garage Cafe and Miles Construction, and was isolated from the neighbouring building by a seismic gap (see Figure 5.17(d)). The original structural system consisted of load bearing external URM walls with timber diaphragms and a concrete lintel beam running the full length of the building on the Hereford Street and Liverpool Street sides. The street-facing facade walls were perforated URM walls whereas the rear of the building consisted of stiff solid shear walls. The building had a sloping roof and the parapet height varied from zero to about 1 m at the side adjacent to the neighbouring building. From preliminary scratch tests it was established that a lime based weak mortar having coarse aggregate was used in the original construction.

(a) Exterior view

(b) Steel moment frame

(c) Highlighted crack location at the rear of building

(d) Reduction of seismic gap

Figure 5.17 Joe's Garage Cafe (imagaes taken post-September 2010)

The building was seismically retrofitted in 2004 using large steel portal frames oriented in the transverse direction of the building, spaced at approximately 4 m centres as shown in Figure 5.17(b). The building floor plan is shown in Figure 5.18. Diaphragm strengthening was not observed in the interior of the building.

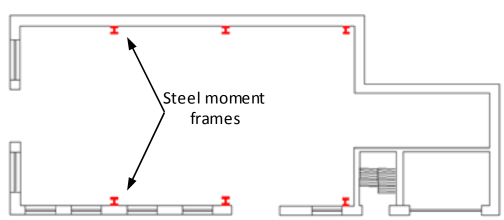


Figure 5.18 Floor plan of Joe's Garage Cafe, showing retrofit

5.4.3 **Welstead House, 184-188 Manchester Street**

Welstead House was originally constructed in 1905 and was a corner building located at the intersection of Manchester Street and Worcester Street. The building was designed in Edwardian Baroque style by architect Robert England with an 800 m² gross floor area (Rothschild, 2010). The building was occupied by seven tenancies in total, and was a standalone two storey clay brick URM building with a regular rectangular plan and no vertical irregularities. A photograph of the building prior to the 4 September 2010 earthquake is shown in Figure 5.19.

Figure 5.19 Welstead House, 184-188 Manchester Street, before the 4 September 2010 earthquake

(a) Corner view **(b) Side view, showing steel anchors**

Figure 5.20 Welstead House, 184-188 Manchester Street, after the 4 September 2010 earthquake

The roof of the building was constructed in three gabled sections, with the parapet enclosing the roof gables and estimated to have a height of 1.6 m. The wall thickness was three leaves, increasing to four leaves at the parapet. The parapet was secured by ~~a single~~ through anchor plate at the ~~apex~~ of each roof gable (i.e. a total of three anchors on the Manchester Street side). A concrete frame was placed at the bottom floor level to allow for large open shop fronts.

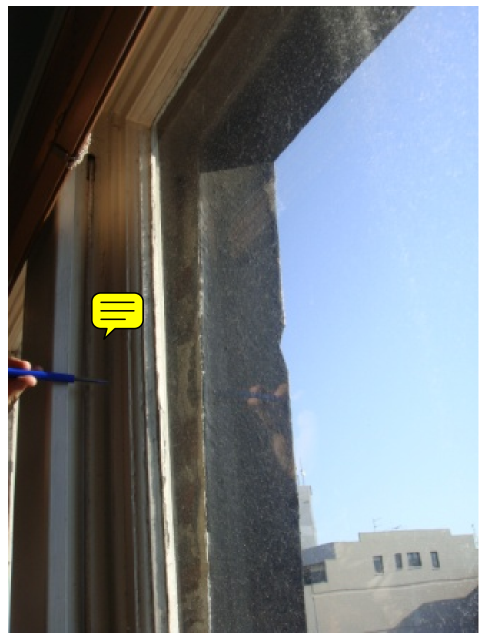
The building experienced a complete out-of-plane collapse of the street front corner facade walls (see Figure 5.20(a)). Anchors in the gables did not provide sufficient restraint, as they remained in the timber roof structure following the earthquake as shown in Figure 5.20(b). Steel anchor plates which were observed along the Worcester Street roof were positioned between the masonry leaves. These anchors remained in the timber roof structure, indicating that insufficient out-of-plane restraint was provided. Due to excessive damage and safety considerations the building was demolished following the 4 September 2010 earthquake.

5.4.4 Caxton Press, 113 Victoria Street

The Caxton Press building was thought to have been constructed in the 1870's. The building was a two storey isolated building that was surrounded on two sides by a reinforced concrete block building as shown in Figure 5.21(a). The Caxton Press building was formerly a bakery, with the baker's oven still intact behind the modern plasterboard walls. The side walls are solid two leaf walls constructed using English bond, which has alternating header and stretcher courses, whereas the facade wall has no visible header courses.

The ground floor street-front was open, accommodating the placement of circular cast-iron columns to support the upper storey walls. The timber diaphragm joists span parallel to the facade wall, with the floorboards running perpendicular.

(a) Exterior view



(b) facade wall pulled away from the side walls

Figure 5.21 Caxton Press building at 113 Victoria Street (images taken post-September 2010)

The Caxton Press building was extensively damaged during the Darfield earthquake and the subsequent aftershocks. From external observation, the parapets on the facade wall had collapsed, the top of the gabled side walls had failed due to out-of-plane loading seen in Figure 5.21(a), the perforated facade wall had developed extensive shear cracks through the spandrel over the openings, and the facade wall had pulled away from the side walls due to insufficient anchorage, as shown in Figure 5.21(b). Furthermore, pounding was evident from cracking on the side walls adjacent to where the new concrete block building butted up to the URM building. On internal inspection, evidence of diaphragm movement was apparent as indicated by displacement of the floor boards and the 15 mm displacement of the bricks in the side walls.

The building owner, who was standing outside the building at the time of the first major aftershock, recalls seeing the brick wall move in a wave pattern, which indicates possible diaphragm movement and weak cohesion between the bricks and mortar. The building was demolished following the 4 September 2010 earthquake.

5.4.5 **Cecil House / Country Theme Building, 68-76 Manchester Street**

The Cecil House / Country Theme building was an "L" shaped corner building located at 68-76 Manchester Street, on the corner of St Asaph and Manchester Streets (see Figure 5.22(a)). The building had two stories, was constructed in 1877 in the neo-classical style, and was believed to have significantly contributed to the heritage value and character of the Commercial Urban Conservation Area (Opus International Consultants, 2005).

(a) Corner view showing parapet collapse **(b) concrete beam on the ground**

Figure 5.22 Cecil House / Country Theme Building, 68-76 Manchester Street (images taken post-September 2010)

The front façade of the building was a three leaf clay brick URM wall, with two leaf thick parapets located along the street-facing perimeter. The parapet had a poorly reinforced (approximately 6 mm round bars at each corner) concrete beam on top.

The most apparent earthquake damage was the toppled parapets around the street frontage as illustrated in Figure 5.22(b), with a lightly reinforced concrete beam on top of the parapet providing insufficient restraint. Falling parapets landed on the canopies below, overloading the supporting tension braces that caused a punching shear failure in the masonry wall and subsequent canopy collapse. The connections appeared to consist of a long, roughly 25 mm diameter rod, with a round steel plate (about 10 mm thick) at the wall end that was approximately 150 mm in diameter.

No evidence of through anchors connecting the roof diaphragm to the wall structure was observed. Some in-plane damage to the far end of the building along Manchester Street was evident, mostly consisting of cracking through the spandrel and some horizontal cracking through the piers.

The building was partially demolished following the 22 February 2011 earthquake.

Section 6:

Demolition statistics and information on the cost of seismic improvement

This section provides information on building demolitions in Christchurch following the 2010/2011 Canterbury earthquake swarm, followed by details associated with the costs of seismic improvement of unreinforced masonry (URM) buildings. It is shown that the majority of demolished buildings were constructed of URM and that the cost of seismic improvement of the national URM building stock exceeds the current value of this building stock.

6.1 Christchurch building demolition statistic

A list of 224 buildings that have been demolished as a result of the 2010/2011 Canterbury earthquake swarm is presented in Appendix C. Figure 6.1 shows that 85% of these buildings were constructed of unreinforced masonry, clearly indicating that this class of building suffered the most extensive damage in the earthquakes.

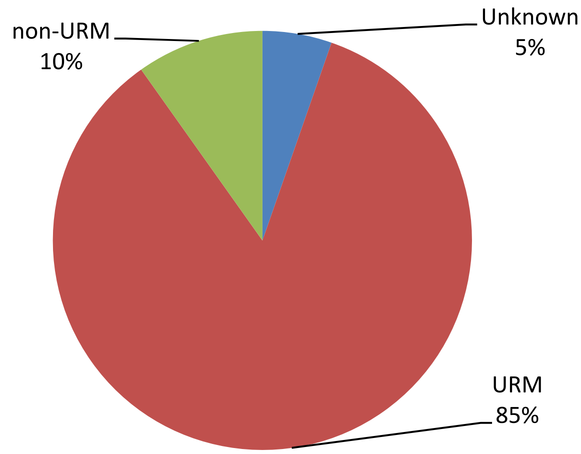


Figure 6.1 Distribution of construction types for 224 demolished buildings in Christchurch

The location of the demolished URM buildings is indicated on a map in Figure 6.2, with Figure 6.3 providing greater detail of the former location of these buildings within the Christchurch Central Business District (CBD).

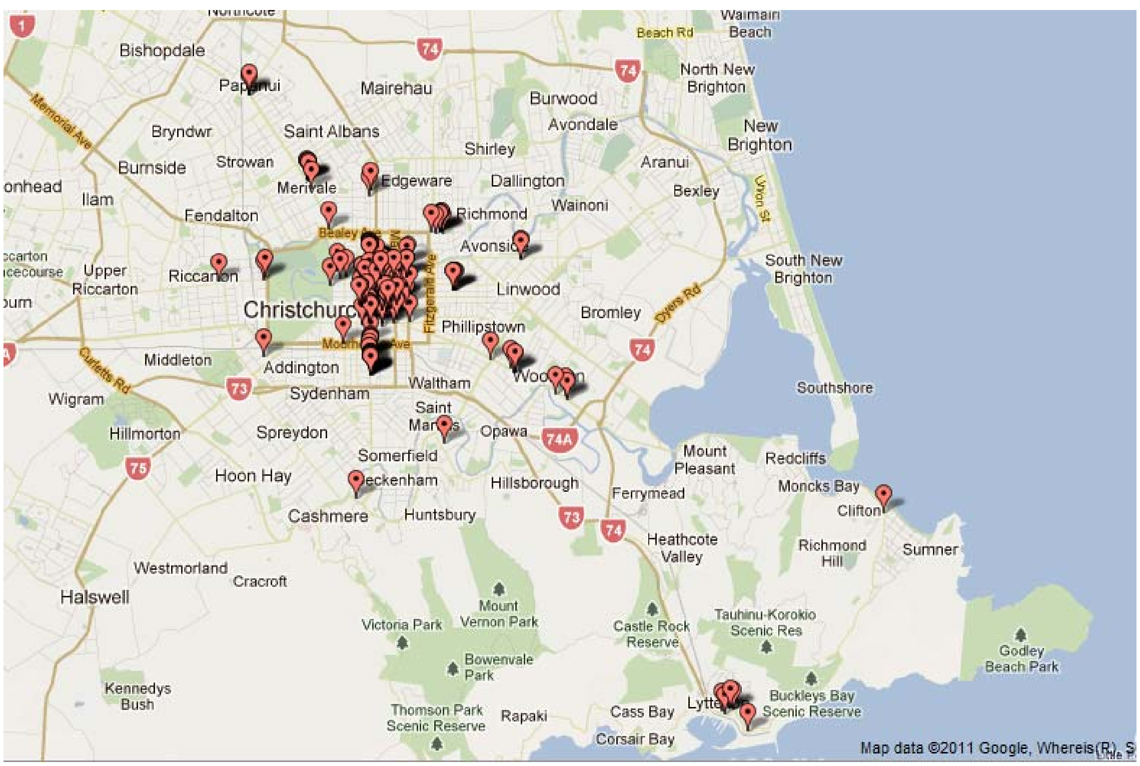


Figure 6.2 Overview of the location of demolished URM buildings (as at 25 July 2011)

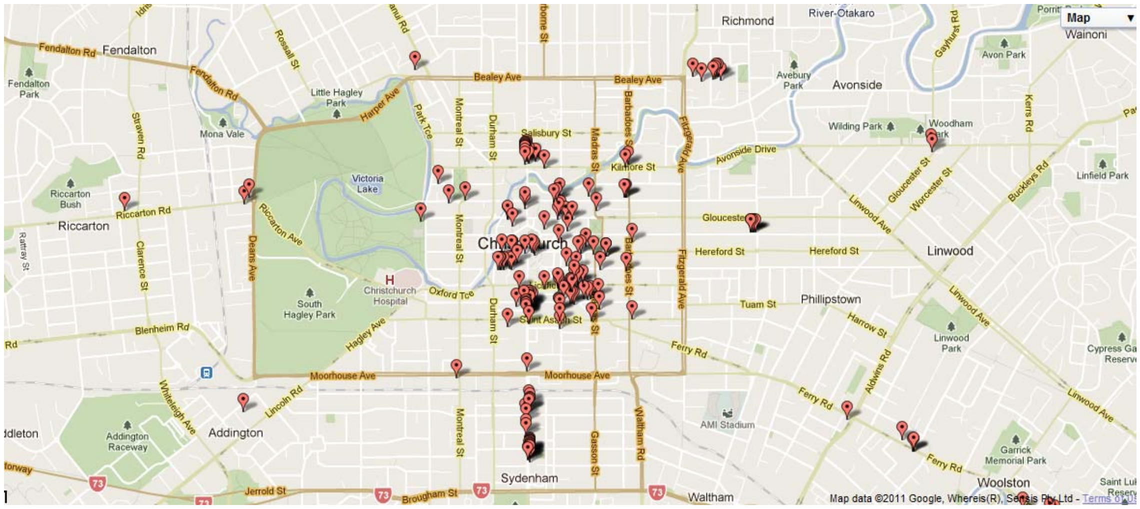


Figure 6.3 Location of demolished URM buildings in the Christchurch CBD (as at 25 July 2011)

Demolitions continue to occur and the data reported in Appendix C and Figure 6.1-Figure 6.3 are for the date up to 25 July 2011. This information will require updating as demolitions continue.

6.2 Costs of seismic improvements

Seismic retrofit cost is a significant factor affecting property owners' decisions to seismically rehabilitate their earthquake prone buildings (EPBs). Egbelakin et al. (2011) revealed that a high cost of retrofitting an EPB is a significant impediment affecting owners' decisions to rehabilitate their EPBs. The New Zealand study conducted by Egbelakin and colleagues revealed that 90% of the interviewees across all the cases studied disclosed that seismic retrofit cost is generally high and can become an economic burden on property owners. Hidden costs associated with retrofitting EPBs were regarded as one of the main contributors to the high cost of retrofitting EPBs (EERI, 2003), resulting in difficulty when attempting to accurately estimate the overall cost of retrofitting EPBs. Hidden costs relate to expenditure that cannot be estimated until the rehabilitation work commences or is completed (Bradley et al., 2008) and are characterised by several variations that depend on factors such as location, type of structure, building characteristics, rehabilitation scheme, the performance standard desired and other work(s) relating to the provisions in the building code that are triggered by the decision to retrofit. Both direct costs (seismic and non-seismic retrofit construction cost) and indirect costs (costs due to business disruption, loss of revenue) associated with seismic retrofit further complicate the cost estimation process (Bradley et al., 2008).

One way to overcome issues relating to seismic retrofit cost is to develop a strategy that will incorporate the seismic retrofit cost into a larger upgrade i.e. implementing seismic improvements during an on-going facility management program (EERI, 2003).

Teamwork during the conceptual design stage in a rehabilitation project can also reduce cost, as all stakeholders can discuss and evaluate cost cutting measures (EERI, 2000).

A motivating factor that could enhance property owners' decisions to invest in seismic retrofitting is the likelihood of cost recovery through increased rents or profits at the time of sale. However Egbelakin et al. (2011) found that cost recovery from retrofitted EPBs is difficult as the money expended on rehabilitation does not increase the market competiveness of the building. Egbelakin and colleagues specifically found that 92% of the owners of EPBs could not recoup any financial benefits from their investments on seismic retrofitting~~; with~~ ~~only~~ 10% of the owners elucidated that ~~although~~ the investment ~~is~~ prohibitive at the time of retrofitting; implementing seismic retrofit could help to save cost associated with future ~~rehabilitation~~ and minimises business disruption ~~due to possible changes in regulation.~~ Likewise, Lindell & Perry (2004) highlighted that substantial financial aid and low-interest loans to owners of EPBs were significant motivators for improved seismic retrofit implementation.

6.3 Cost of seismic improvement of the national URM building stock

Christchurch City Council has published information on the projected cost of seismic improvement of URM buildings[16]. This document identifies that the cost to strengthen a typical URM building to 33% NBS is in the range of $350-450/m². As reproduced in Table 6.1, Christchurch City Council have also published data on the projected costs to strengthen 295 URM buildings to 33% NBS and to 67% NBS.

Table 6.1 Christchurch City Council Listed Buildings (25 March 2010)

Method of construction	Heritage Significance 1 City Plan GP 1 and BPDP HPT Cat 1	Heritage Significance 2 City Plan GP 2	Heritage Significance 3 City Plan GP 3 and 4, BPDP HPT Cat 2 and Notable	TOTAL	Strengthening Cost (to 33%) (million)	Strengthening Cost (to 67%) (million)
Unreinforced masonry	55	70	170	295	$137	$344
Reinforced concrete	1	7	21	29	$23	$57
Timber framed and other	18	19	126	163	$9	$22
TOTAL	74	96	317	487	$169	$421
Additional cost of fire and disabled access requirements					20%-100%	60%-160%

6.3.1 Approximate cost of seismic improvement of national URM building stock

The accurate determination of costs for the seismic improvement of the national URM building stock requires expertise in quantity surveying. The authors acknowledge that they have no such expertise, but nevertheless present the following analysis based upon

[16] **REVIEW OF EARTHQUAKE-PRONE, DANGEROUS AND INSANITARY BUILDINGS POLICY**:
http://www1.ccc.govt.nz/council/proceedings/2010/march/regplanning4th/1.reviewofearthquakepronedangerousinsanitarybuildings.pdf

data presented at various locations throughout this report in order to trigger dialogue on the subject.

From Table 6.1 it may be determined that the cost of improving the identified Christchurch URM buildings to 33%NBS is M$137 and that the cost to instead improve these buildings to 67%NBS is M$344. Consequently it may be determined that the cost of improving to 67%NBS has a factor of 344/137 = 2.51.

Figure 2.11(b) shows that there are approximately 1376 URM buildings nationwide having a strength of less than 33% NBS and 2008 URM buildings nationwide having a strength of 34-67% NBS. It is recognised that there is uncertainty in these numbers and so therefore no attempt has been made to reduce the building count in accordance with the demolition data reported in section 6.1 and Appendix C. Section 2.4 reports that the URM buildings extracted from the QV database had a total floor area of 2,100,000 m^2. Consequently this data can be combined as shown in Table 6.2 to suggest an indicative cost of improving the national URM building stock to 67% NBS. In this analysis a typical cost of $450/$m^2$ to elevate to 33%NBS is assumed in order to partially compensate for inflation during the period March 2010 to July 2011.

Table 6.2 Projected cost of seismically improving the national URM building stock to 67% NBS

Current strength (% NBS)	Number	Total Floor Area (1,000,000 m^2)	Cost ($/$m^2$)	M$
0-33	1376	0.748	1129.	844.9
34-67	2008	1.090	450	1231.2
>68	483	0.262	-	-
Total	3867	2.100		2076.1

Note that the estimated value to improve the national URM building stock to 67% NBS is approximately $2.1 billion. This number can be compared with the estimated value of these buildings of approximately $1.5 billion, as reported in Table 2.3.

Section 7:

Recommendations and closing remarks

7.1 Recommendations

1. ~~Identify~~ all unreinforced clay and stone masonry building stock in New Zealand[17].
   - Unreinforced masonry buildings consistently perform poorly in large earthquakes. Previously, not all territorial authorities have had a register of URM buildings located within their jurisdiction. In order to ensure that all URM buildings in New Zealand do not pose a safety risk to the public, it is essential that the presence and location of these buildings are known.
2. Successful retrofits showed that ~~it is possible to make strengthened URM buildings survive severe earthquake ground motion~~.
3. There are several logical stages of building performance improvement that should be considered. The number of stages involved for seismic retrofitting of ~~a~~ building will depend upon how ~~well~~ the building owner and/or officials and occupants want the building to ~~behave~~.
   - 1st stage: ensure public safety by eliminating falling hazards. This is done by securing/strengthening URM building elements that are located at height (eg, chimneys, parapets, ornaments, gable ends).
   - 2nd stage: strengthen masonry walls to prevent out-of-plane failures. This can be done by adding reinforcing materials to the walls and by installing connections between the walls and the roof and floor systems at every level of

[17] In all cases the term URM is used in this section to refer to unreinforced masonry buildings constructed of both clay brick and of stone, or of a combination of the two masonry materials.

the building so that walls no longer ~~respond as vertical cantilevers secured only at their base~~.

- 3rd stage: ensure adequate connection between all structural elements of the building so that it responds as a cohesive unit rather than individual, isolated building components. In some situations it may be necessary to stiffen the roof and floor diaphragms, flexurally strengthen the masonry walls and provide strengthening at the intersection between perpendicular walls.
- 4th stage: if further capacity is required to survive earthquake loading, then the in-plane shear strength of masonry walls can be increased or high-level interventions can be introduced, such as the insertion of steel and/or reinforced concrete frames to supplement or take over the seismic resisting role from the original unreinforced masonry structure.

4. The authors propose that all URM buildings should go through the first two stages of building improvement so that the targeted structural elements have their strength elevated to match that required for equivalent structural elements in a new building located at the same site. For 3rd and 4th stage improvements, building strengthening should aim for 100% of the requirement for new buildings but as a minimum, 67% might be acceptable.
5. Recommendation 4 should be a national requirement, rather than being left to territorial authorities to draft and monitor their own individual policies.
6. There is a need for more widespread technical capability for seismic assessment (analysis) and design of URM buildings in the New Zealand engineering community.
7. In view of the uncertainties regarding the seismic strength of existing URM buildings, it is recommended that field testing be conducted on some of the URM buildings in Christchurch that are scheduled for demolition.
8. Budgeting constraints will likely limit the extent to which URM buildings can be seismically upgraded. Therefore priority should be given to ensuring public safety by implementing Recommendation 3: Stage 1 and Stage 2 as soon as possible for all URM buildings.

7.2 Closing Remarks

1. There were no surprises amongst the collapse mechanisms observed in URM buildings.
2. Current building standards are appropriate and are representative of 'world's best practice'.
3. The amplitude of ground shaking experienced by URM buildings in Christchurch was well in excess of that prescribed by the current design spectra for Christchurch buildings located on soft soils. Nevertheless, well considered, conceived and implemented seismic retrofits of URM buildings performed well, even when the building experienced ground motion that ~~was~~ well in excess of the design level for Christchurch.
4. The URM building damage statistics were significantly worse after the 22nd February 2011 earthquake than they were after the 4th September 2010

earthquake due to the severity of local ground motions in the CBD during the 22 February earthquake.

5. The estimated cost to upgrade all 3867 URM buildings in New Zealand to a minimum of 67% of the NBS is roughly $2.1 billion, which is more than the estimated total value of the URM building stock of $1.5 billion. However, a multi-stage retrofit improvement program has been recommended and it is anticipated that the cost of implementing stage 1 and stage 2 improvements will not be excessive and should be within the budget capability of most building owners.

Section 8:

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Appendix A: [18]

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:

[18] 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 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:

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.

**SR 1983/225*

Appendix B:

Estimation of URM building population and distribution

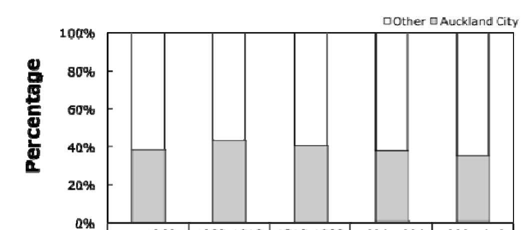
Several sources of data were utilised for estimating the number of URM buildings in existence throughout the country: the official population data of New Zealand between 1900 and 1940 (Census and Statistics Office, 1890–1950), a survey of potentially earthquake prone commercial buildings in Auckland City conducted by Auckland City Council in 2008 in conjunction with the research team, and data provided by Wellington City Council and Christchurch City Council.

In surveying potentially earthquake prone commercial buildings in Auckland City, a total of 1335 buildings were identified to have been constructed before 1940. Although buildings with a construction date up to and including 2007 were surveyed, very few URM buildings were found to have been built in Auckland City after 1940. Therefore, only pre-1940 buildings were considered. Of the 1335 buildings, 28.9% were URM, 35.3% were timber, 16.3% were comprised of reinforced concrete frame and brick infill, 1.1% were reinforced masonry, 17.8% were reinforced concrete frame or shear wall buildings and 0.6% were moment resisting steel or braced steel buildings. Using the associated construction date of each building the total sample was grouped according to decade. Pre-1900 was considered as a single grouping. Table B.1 shows the number of buildings identified in the survey according to their construction date.

Table B.1 Auckland City pre-1940 potentially earthquake prone buildings

	Pre-1900	1901-1910	1911-1920	1921-1930	1931-1940	Total	Percentage
URM	6	24	16	277	63	385	28.9%
Timber	3	21	16	341	90	417	35.3%
Brick infill	4	13	4	123	74	217	16.3%
Reinforced masonry	0	0	0	10	5	15	1.1%
Reinforced concrete	1	7	7	152	71	238	17.8%
Steel	0	0	0	5	3	8	0.6%
Total	15	65	45	907	304	1335	100%

In order to estimate the number of URM buildings in other parts of the country, the data from Auckland City Council were extrapolated using official population data. In the late 19th and early 20th Century, New Zealand was divided into the following provinces: Auckland, Taranaki, Hawkes Bay, Wellington, Marlborough, Nelson, Canterbury and Otago-and-Southland. Auckland Province was made up of the area of the North Island from Taupo and north (everywhere which currently celebrates Auckland Anniversary Day) (Census and Statistics Office, 1890–1950). Consequently, the area over which Auckland City Council has jurisdiction in 2009 is only a part of the former Auckland Province, and the current boundaries of this jurisdiction are equivalent to that of the Eden County up until 1940. This county historically included the boroughs of Auckland City, Mt Albert, Mt Eden, Newmarket, Parnell, Onehunga, Grey Lynn, One Tree Hill, and also Ellerslie Town District. The proportion of the population of the historic Auckland province which is made up by the current Auckland City was found using the population data from official New Zealand Year Books (Census and Statistics Office, 1890–1950). The average population of Auckland City and other parts of Auckland Province are shown in Figure B.1.



	pre-1900	1900-1910	1910-1920	1920-1930	1930-1940
Other	108,660	109,513	166,260	245,717	336,570
Auckland City	67,278	84,068	112,096	147,922	180,297

Figure B.1 Proportion of population in the former Auckland Province living in the equivalent current Auckland City

Using the same proportional relationships shown in Figure B.1, the number of currently existing URM buildings in the historic Auckland Province was estimated based on the number of currently existing URM buildings in Auckland City. For example, in the decade 1900–1910, Auckland City made up 43% of the population of Auckland Province. It is assumed that building prevalence was approximately proportional to population and that the rate of building demolition has been uniform throughout the former Auckland

Province. There are 24 URM buildings identified from that decade now existing in Auckland City, and assuming these also make up 43% of the total number of buildings in the historic Auckland Province, then there are 55 existing URM buildings which were built between 1900 and 1910 in the whole of the equivalent Auckland Province today. Similarly, an indicative URM-buildings-per-capita ratio is determined. These data are summarised in Table B.2, clearly showing that the majority of URM buildings were constructed in the decade 1920 – 1930.

Table B.2 Population data and URM buildings for Auckland City and Auckland Province

	Pre-1900	1901-1910	1911-1920	1921-1930	1931-1940
Population of former Auckland Province	175,938	193,581	278,357	393,639	516,886
Population of equivalent current Auckland City	67,278	84,068	112,096	147,922	180,297
Proportion Auckland City/Province	38.2%	43.0%	41.1%	37.5%	35.2%
Actual current Auckland City URM buildings	6	24	16	277	63
Estimated current Auckland Province URM buildings	16	55	40	737	178
Estimated current URM buildings per 100,000 people	9.1	28.4	14.4	187.2	34.4

In addition to the data provided from Auckland City Council and extrapolated to estimate the number of URM buildings in the historic Auckland Province, similar methods were used to extrapolate the data provided by Wellington City Council and Christchurch City Council. Based on official provincial populations of the time, the number of URM buildings currently remaining in the historic provinces of Taranaki, Marlborough, Nelson and Westland were also estimated assuming the same ratio of URM buildings per 100,000 people as in Auckland Province, as in the absence of specific data there is believed to be no evidence available to suggest that the ratio of URM buildings per 100,000 people in Auckland is not valid for these provinces.

Based on evidence provided in Hopkins (2009), it was considered inappropriate to assume a similar buildings per capita ratio as in Auckland for the remaining provinces of Hawke's Bay and Otago-and-Southland. When legislative guidance was introduced in 1968 (New Zealand Parliament, 1968) for assessing and upgrading earthquake prone buildings, Auckland and Wellington City Councils took a strong interest in strengthening URM buildings whilst Christchurch and Dunedin City Councils took a more passive approach to implementing the legislation. Consequently, the rate of seismic retrofit and/or demolition and reconstruction in Auckland and Wellington was significantly different from that in Dunedin and Christchurch. Dunedin is the largest city in the former Otago-and-Southland Province and its rate of redevelopment was assumed to be characteristic of the whole province. Consequently, the number of buildings remaining in Otago-and-Southland was estimated using the buildings per 100,000 people ratio of Canterbury.

The 1931 M7.1 earthquake in Hawke's Bay destroyed a significant number of URM buildings in the Hawke's Bay Province, especially in Napier. As a consequence of this and the resulting awareness of the vulnerability of URM buildings, the number of remaining buildings in Hawke's Bay can be expected to be less than what would be estimated using the relationships outlined above. Nevertheless, there is no data available on the actual number of URM buildings in Hawke's Bay, and because of this, the ratio of URM buildings per 100,000 people in Hawke's Bay was estimated to be half that of Auckland's. The estimated number of existing URM buildings in each province (calculated prior to the 2010/2011 Canterbury earthquake swarm) is shown in Table 2.2 and in Figure B.2, and the construction date of URM buildings nationwide is shown in Figure B.3, and are grouped according to the first year in each decade. This information again shows that the majority of existing URM buildings nationwide derive from the decade 1920–1930.

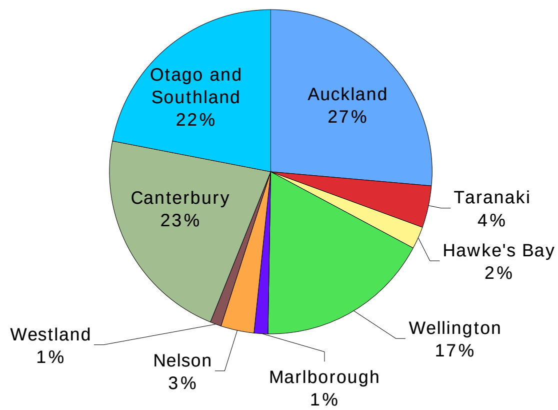


Figure B.2 Estimated provincial populations of URM buildings (data compiled prior to 22 September 2010)

It is acknowledged that the data presented here are useful primarily as an initial estimation only and may not accurately represent the number of URM buildings in other regions outside of Auckland, especially in smaller towns. The number of buildings from a particular decade in Auckland captures only those buildings which still exist, rather than all the buildings which were constructed in that time period, and the rate of demolition and redevelopment in Auckland City may not be representative of the comparable rate in other parts of the country. Whereas in Auckland economic factors may have provided a stimulus for demolition of older URM buildings and development of newer structures, this may have not been the case in smaller towns. Smaller cities such as Wanganui, Timaru and Oamaru did not receive equivalent levels of investment and development in the 1960s and 1970s for economic reasons, and consequently many old buildings which would have otherwise been demolished in that time period still exist now (McKinnon, 2008). Moreover, legislation governing the seismic performance of existing buildings may have resulted in different rates of development. For example, Blenheim is in a higher seismic zone (Z = 0.33) than New Plymouth (Z = 0.18) and if a building in Blenheim which was determined to be earthquake risk and subsequently demolished was instead situated in New Plymouth, because of the lower seismicity, it may have been found to not be earthquake risk. Finally, this is not an estimation of the number of earthquake prone buildings in New Zealand, apart from the inference that many URM buildings are likely to meet the criteria of being earthquake prone.

In addition to the above estimate of the number of URM buildings in New Zealand, data on the New Zealand building stock were obtained from Property IQ, a part of Quotable Value Ltd (QV), which is a valuation and property information company in New Zealand. QV collects building information and conducts building valuations for rating purposes for most New Zealand Territorial Authorities. In the council valuation data, the building material and age (decade), among other data elements, is recorded. The building material refers to the wall cladding and is not a comment on the load carrying materials of the structure. It was assumed that no URM buildings were constructed in New Zealand after 1950 (Stacpoole & Beaven, 1972) and that buildings with a brick veneer but other materials for the load bearing parts of the structure (for example, timber frame buildings with a brick veneer) are recorded as "mixed materials" in the database. All entries for buildings constructed in New Zealand before 1950 and with "brick" recorded as the cladding description in the QV database were extracted. While it is acknowledged that a cladding description recorded as "brick" can include brick, brick veneer, adobe and rammed earth as the material type, it was considered that such an extraction of data would be a legitimate reflection of the URM building stock in New Zealand.

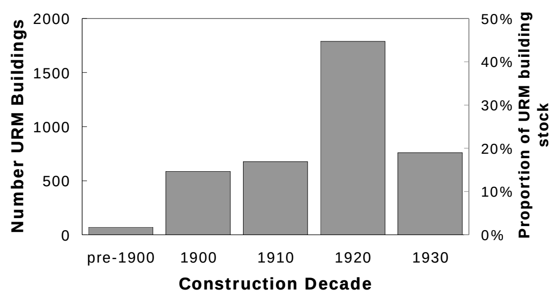


Figure B.3 Construction decade of URM buildings in New Zealand

These records were analysed according to construction date, building height and financial value. Table B.3 shows the decade in which each URM building was built. Brick buildings with mixed age are entered on the QV database as pre-1950, but their exact age is indeterminate from the data recorded. The number of URM buildings with a confirmed construction date are shown in Figure B.4, and are grouped according to the first year in each decade.

Table B.3 Number of URM buildings from QV according to construction decade

Decade	URM Buildings
1871 – 1880	43
1881 – 1890	23
1891 – 1900	71
1901 – 1910	469
1911 – 1920	646
1921 – 1930	878
1931 – 1940	514
1941 – 1950	218
Mixed	725
Total	3589

Figure B.4 clearly shows a trend where the number of URM buildings initially increased until the end of the 1920s, and subsequently declined. This trend follows the increasing rate of European immigration and associated infrastructure development in New Zealand in the early 20th Century, until the 1931 M7.8 Hawke's Bay earthquake, after which URM was no longer considered a favourable building material.

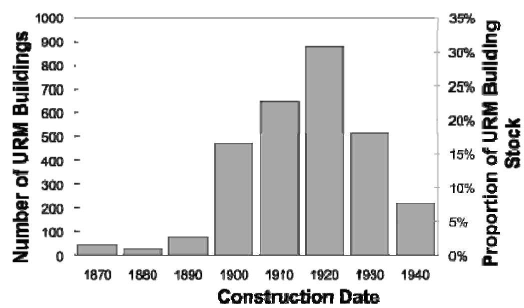


Figure B.4 Number of URM buildings from QV according to construction decade

Appendix C:

List of demolished buildings

Table C.1 reports the details of buildings in Christchurch that have been demolished following the 2010/2011 Canterbury earthquake swarm. 224 buildings are reported in Table C.1.

Table C.1 Christchurch building demolished following the 2010/2011 Canterbury earthquake swarm (as at 25 July 2011)

No.	Street	Construction Type	Status	Property
240	Armagh Street	non-URM	Non-Heritage	Amicus House Residential 16
32	Armagh Street	URM	Heritage	Christ's College - Cranmer Centre (ex-ChCh Girls
52	Armagh Street	URM	Heritage	Windsor Hotel
182	Armagh Street	URM	Non-Heritage	Chen's Kitchen Grand Total 102
195	Armagh Street	URM	Non-Heritage	Music Institute Commercial 86
245	Armagh Street	URM	Non-Heritage	Hairdresser
247	Armagh Street	URM	Non-Heritage	Laundrette
249	Armagh Street	URM	Non-Heritage	Dairy and Sinbad Foods
184-186	Armagh Street	URM	Non-Heritage	Tax Link & Yumi Sushi
272	Barbadoes Street	URM	Non-Heritage	Frauenreisehouse Women's Hospital
21	Bealey Avenue	URM	Heritage	Carlton Hotel (Legally 1 Papanui Rd)
18	Bedford Row	URM	Non-Heritage	
167	Bowhill Road	non-URM	Non-Heritage	Fish & Chip Shop
137	Caledonian Road	unknown	Non-Heritage	House & Garage
35	Cambridge Terrace	non-URM	Non-Heritage	Rolleston Courts Apts

82	Cashel Street	URM	Non-Heritage	The Bog & The Vault
86	Cashel Street	URM	Non-Heritage	Trade Aid
88	Cashel Street	URM	Heritage	Cafe Blue
94	Cashel Street	URM	Non-Heritage	Last Train to India
109	Cashel Street	URM	Heritage	Cashel Mall Block (Former Press & Weekly Press Building
116	Cashel Street	URM	Non-Heritage	Flight Centre
181	Cashel Street	URM	Non-Heritage	Sushi Q, Cashel Liquor Centre, Cashel Convenience
236	Cashel Street	URM	Heritage	St Paul's Church
274	Cashel Street	URM	Heritage	The Provincial
112-112a	Cashel Street	URM	Non-Heritage	Acquisitions / Eden Alley / Harris Dental Ltd
208-210	Cashel Street	URM	Non-Heritage	Enabling Better Business / Comcare Trust
2	Cashmere Road	unknown	Non-Heritage	4 x Rental Units
1/8	Cashmere Road	URM	Non-Heritage	Cashmere Seafood-fish&chip and New Just Thai
32	Cathedral Square	URM	Heritage	The Press Building
53	Cathedral Square	URM	Heritage	Chancery Chambers
2	Chester Street	URM	Heritage	Stratham House - Cathedral Grammar
6	Circuit Street	unknown	Heritage	Elizabeth House
992	Colombo Street	unknown	Non-Heritage	
382	Colombo Street	URM	Non-Heritage	The Great Opportunity Shop & Dairy
386	Colombo Street	URM	Heritage	Antiques and Collectables
388	Colombo Street	URM	Heritage	Sydenham Book Exchange
390	Colombo Street	URM	Heritage	Triton Dairy
392	Colombo Street	URM	Heritage	Image Photo & Frame
394	Colombo Street	URM	Heritage	Image Photo & Frame
398	Colombo Street	URM	Heritage	Sydenham Stationary
400	Colombo Street	URM	Heritage	
402	Colombo Street	URM	Heritage	
404	Colombo Street	URM	Heritage	Ascot TV
406	Colombo Street	URM	Heritage	Ascot TV
439	Colombo Street	URM	Non-Heritage	
441	Colombo Street	URM	Non-Heritage	Churchill Tavern
457	Colombo Street	URM	Non-Heritage	Vacuum Cleaner Repairs
480	Colombo Street	URM	Non-Heritage	Rob Roys Scottish Bar
482	Colombo Street	URM	Non-Heritage	
484	Colombo Street	URM	Non-Heritage	Change of Status to previous release
490	Colombo Street	URM	Non-Heritage	Metro Imports
494	Colombo Street	URM	Non-Heritage	Kashmir Building
590	Colombo Street	URM	Non-Heritage	Joyful Restaurant & adjacent Bakery
592	Colombo Street	URM	Non-Heritage	

593	Colombo Street	URM	Non-Heritage	Southern Ink
595	Colombo Street	URM	Non-Heritage	Lotus Heart
597	Colombo Street	URM	Non-Heritage	Original Haircuts
599	Colombo Street	URM	Non-Heritage	Sushi Dining Kinji
615	Colombo Street	URM	Heritage	Austral Building
618	Colombo Street	URM	Non-Heritage	Adult Cash Discounter
620	Colombo Street	URM	Non-Heritage	Falconer's Shoe Store
622	Colombo Street	URM	Non-Heritage	Computer Centre
624	Colombo Street	URM	Non-Heritage	
626	Colombo Street	URM	Heritage	Bean Bags & Beyond
773	Colombo Street	URM	Non-Heritage	Bettys Liquor Store
783	Colombo Street	URM	Non-Heritage	Metro CafÄ
789	Colombo Street	URM	Non-Heritage	The Orange Tree, Footprints Organic CafÄ
800	Colombo Street	URM	Non-Heritage	Sala Thai
801	Colombo Street	URM	Non-Heritage	Dusty Old Things Antiques
803	Colombo Street	URM	Non-Heritage	The Painted Room
805	Colombo Street	URM	Non-Heritage	Kim's Restaurant
807	Colombo Street	URM	Non-Heritage	Kildonan House
809	Colombo Street	URM	Non-Heritage	Studio Works
811	Colombo Street	URM	Non-Heritage	CafÄ Valentino Restaurant
813	Colombo Street	URM	Non-Heritage	CafÄ Valentino Restaurant
815	Colombo Street	URM	Non-Heritage	CafÄ Valentino Restaurant
819	Colombo Street	URM	Non-Heritage	Phu Thai
1/492	Colombo Street	URM	Non-Heritage	Modern Engravers
1049-1047	Colombo Street	URM	Heritage	St Albans Community Centre
2/492	Colombo Street	URM	Non-Heritage	Speedway Bookshop
380A	Colombo Street	URM	Non-Heritage	Tasty Tucker Bakery
384-384A	Colombo Street	URM	Non-Heritage	Fish'n'Chips & Eve's Gifts
461-469	Colombo Street	URM	Heritage	Storage Sheds
595A	Colombo Street	URM	Non-Heritage	Billiken Restaurant
597A	Colombo Street	URM	Non-Heritage	Longhorn Leather Shop
599A	Colombo Street	URM	Non-Heritage	Hi Tech Books
601-601A	Colombo Street	URM	Non-Heritage	Pleasure Plus, Longhorn Leather Shop
753-759	Colombo Street	URM	Heritage	2-storey commercial
803a	Colombo Street	URM	Non-Heritage	The Painted Room
804-806	Colombo Street	URM	Non-Heritage	
808-812	Colombo Street	URM	Non-Heritage	Gallery 810, Welcome Dairy, Bodhi Tree
159	Deans Avenue	URM	Non-Heritage	Hunter Lounge Suites
1/462	Durham Street	non-URM	Non-Heritage	
2/462	Durham Street	non-URM	Non-Heritage	
3/462	Durham Street	non-URM	Non-Heritage	

188-192	Ferry Road	unknown	Non-Heritage	Restaurant Schwass / Footstep Shoe Repairs
360	Ferry Road	URM	Non-Heritage	
454	Ferry Road	URM	Non-Heritage	Yazu Hair Design
455	Ferry Road	URM	Non-Heritage	Dowsons Shoes
580	Ferry Road	URM	Heritage	A&T Burt Building (former Nugget Factory)
628	Ferry Road	URM	Non-Heritage	Big Eds Takeaways
689	Ferry Road	URM	Heritage	
697	Ferry Road	URM	Heritage	Ferry Road Law Centre
452A	Ferry Road	URM	Non-Heritage	Tan's Chinese Takeaways
454A	Ferry Road	URM	Non-Heritage	St. Martins Pottery
215	Fitzgerald Avenue	unknown	Non-Heritage	
97	Fitzgerald Avenue	non-URM	Non-Heritage	Block Wall on Boundary
466	Gloucester Street	unknown	Non-Heritage	Boarding House
192	Gloucester Street	non-URM	Non-Heritage	The Clinic
198	Gloucester Street	non-URM	Heritage - Significant	TVNZ Building
241	Gloucester Street	non-URM	Non-Heritage	Stonehurst Backpackers
94	Gloucester Street	URM	Heritage	The Garage
96	Gloucester Street	URM	Heritage	Gusto Beijing Duck
173	Gloucester Street	URM	Non-Heritage	Map World, City Fish & Chips, McCammon Dairy and Bebols
174	Gloucester Street	URM	Non-Heritage	Tulsi
194	Gloucester Street	URM	Heritage	Wave House (Old Winnie Bagoes)
701	Gloucester Street	URM	Non-Heritage	T Bakery
703-709	Gloucester Street	URM	Non-Heritage	
5	Heaton Street	unknown	Non-Heritage	House and Garage
47	Hereford Street	non-URM	Heritage - Significant	St Elmos Courts
190-192	Hereford Street	non-URM	Heritage - Significant	Kenton Chambers
84	Hereford Street	URM	Heritage	Mythai (former NZ Trust and Loan Building)
104	Hereford Street	URM	Non-Heritage	Yorkshire House - Poppy Thai, French Cafe
106	Hereford Street	URM	Non-Heritage	Yorkshire House - Poppy Thai, French Cafe
126	Hereford Street	URM	Non-Heritage	OPSM
134	Hereford Street	URM	Heritage	Hanafins Camera & Video
136	Hereford Street	URM	Heritage	Hanafins Camera & Video
198	Hereford Street	URM	Heritage	Youth Health Centre
202	Hereford Street	URM	Heritage	NZ Prints
203	Hereford Street	URM	Heritage	Avonmore House / Interiors House
234	Hereford Street	URM	Non-Heritage	Church Hall
234	Hereford Street	URM	Non-Heritage	The Vicarage

234	Hereford Street	URM	Heritage	Church of St John the Baptist
170	High Street	URM	Heritage	Head Over Heels
172	High Street	URM	Heritage	Former Knights Butchery
174	High Street	URM	Non-Heritage	Embassy
278	High Street	URM	Heritage	Hanafins Camera & Video
255	Kilmore Street	unknown	Non-Heritage	Octo Ltd
257	Kilmore Street	unknown	Non-Heritage	
132	Kilmore Street	URM	Non-Heritage	Thrifty Car Rental
135	Kilmore Street	URM	Heritage	Caledonian Hall
222	Kilmore Street	URM	Non-Heritage	The Herbal Dispensary
229	Kilmore Street	URM	Heritage	Piko Wholefoods (also known as 359 Barbadoes)
54	Lichfield Street	URM	Non-Heritage	R&R Sport
84	Lichfield Street	URM	Heritage	Fazazz
114	Lichfield Street	URM	Heritage	The Honey Pot CafÄ
115	Lichfield Street	URM	Non-Heritage	Rod Hair Textiles
116	Lichfield Street	URM	Heritage	Ruben Blades
119	Lichfield Street	URM	Non-Heritage	Cotura Fashions
121	Lichfield Street	URM	Non-Heritage	Cotura Fashions
127	Lichfield Street	URM	Non-Heritage	Sound People, I R Thompson & Assoc, The Travel Doctor
6	London Street	URM	Heritage	Mazey Building
9	London Street	URM	Heritage	Empire Hotel
24	London Street	URM	Heritage	Harbourlight Theatre
36	London Street	URM	Heritage	Coastal Living Design Store
38	London Street	URM	Non-Heritage	Lyttleton Fisheries, Fish and Chip Shop
40	London Street	URM	Non-Heritage	Lava Bar
42	London Street	URM	Heritage	Volcano CafÄ
44	London Street	URM	Heritage	The Albion
249	Madras Street	non-URM	Non-Heritage	CTV
271	Madras Street	non-URM	Non-Heritage - Significant	Harcourts Grenadier
192	Madras Street	URM	Heritage	Nurse Maude Building
204	Madras Street	URM	Non-Heritage	Florian Building
268	Madras Street	URM	Heritage	Charlie's Backpackers
253-255	Madras Street	URM	Heritage	Arrow international
11	Main North	URM	Road	Non-Heritage
91-93	Main Road	non-URM	Non-Heritage	Redcliffs Library
87	Manchester Street	URM	Non-Heritage	Beverley Studios
105	Manchester Street	URM	Heritage	H Pannells Boot Emporium
107	Manchester Street	URM	Heritage	Budapest Restaurant
109	Manchester Street	URM	Heritage	John Dary Menswear
110	Manchester Street	URM	Non-Heritage	Nee Hao Asian Delight/Soho/Players/Galaxy Records

204	Manchester Street	URM	Non-Heritage	Iconic Bar
211	Manchester Street	URM	Non-Heritage	Le Plonk
265	Manchester Street	URM	Non-Heritage	Map World, City Fish & Chips, McCammon Dairy and Bebols
293	Manchester Street	URM	Non-Heritage	Subway
141-147	Manchester Street	URM	Non-Heritage	
69-73	Manchester Street	URM	Heritage	Cecil House
20	Marsden Street	unknown	Non-Heritage	
376	Montreal Street	non-URM	Non-Heritage - Significant	Strategy House
192	Moorhouse Avenue	URM	Heritage	Crown Hotel
24	Norwich Quay	URM	Non-Heritage	Lyttelton Hotel
34	Norwich Quay	URM	Heritage	The Royal Hotel
165	Papanui Road	URM	Heritage	Hall
196	Papanui Road	URM	Non-Heritage	Villa Antiques
198	Papanui Road	URM	Non-Heritage	Cookery Nook & Chicotis
203	Papanui Road	URM	Non-Heritage	
204	Papanui Road	URM	Non-Heritage	Kudos hairdrssers
507	Papanui Road	URM	Non-Heritage	Joe Butler Real Estate
509	Papanui Road	URM	Non-Heritage	Memories CafÄ
196A	Papanui Road	URM	Non-Heritage	Love in a Basket
202A	Papanui Road	URM	Non-Heritage	Mansfield Antiques & Momo Sushi
86	Port Hills	non-URM	Road	Non-Heritage Jaishaan Diary
2	Reserve Terrace	URM	Heritage	Time Ball Station
7	Riccarton Road	URM	Heritage	St Christophers Avonhead Bookshop
102A&B	Riccarton Road	URM	Non-Heritage	Computeera Ltd
33D	Rolleston Avenue	URM	Non-Heritage	Christs College (English Block)
244A	Salisbury Street	non-URM	Non-Heritage	Flats
310	St Asaph	URM	Street	Non-Heritage
270	St Asaph Street	URM	Non-Heritage	Southlander Bar
33	Stoke Street	unknown	Non-Heritage	
1	Sumner Road	URM	Heritage	Former Library
160	Tuam Street	URM	Non-Heritage	Canterbury Music Planet
178	Tuam Street	URM	Heritage	Chillis - Also known as 622 - 624 Colombo St
180	Tuam Street	URM	Heritage	
217	Tuam Street	URM	Non-Heritage	Atami Bath House
221	Tuam Street	URM	Non-Heritage	Portobello
223	Tuam Street	URM	Non-Heritage	Global Fabrics / Edward Gibbons
230	Tuam Street	URM	Heritage	Edison Hall (Workshop, Witchery)
232	Tuam Street	URM	Heritage	Domo and Witchery
236	Tuam Street	URM	Heritage	Domo
50	Victoria Street	non-URM	Non-Heritage - Significant	NZ College of Early Childhood Education
167	Victoria Street	non-URM	Non-Heritage	Significant Fidelity House
3	Wades Avenue	URM	Non-Heritage	St Martins Library

16	Wakefield Avenue	URM	Non-Heritage	Sumner Community Centre
92	Wilsons Road	non-URM	Non-Heritage	New World St Martins
14	Wise Street	URM	Heritage	Addington Flour Mil-Grain Store Building
378	Worcester Street	non-URM	Non-Heritage	Shops on Street front
143	Worcester Street	URM	Heritage	Lonsdale House - Gopals + Pedros
387	Worcester Street	URM	Non-Heritage	
389	Worcester Street	URM	Non-Heritage	Wicks Fish Supply
391	Worcester Street	URM	Non-Heritage	
395	Worcester Street	URM	Non-Heritage	Marcels Picnic
393A	Worcester Street	URM	Non-Heritage	Chemist Shop