

Quantifying Unreinforced Masonry Seismic Risk and Mitigation Options in Victoria, BC

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Abstract

Despite their well-known seismic vulnerability, unreinforced masonry (URM) buildings continue to be a leading source of loss of life and property damage in earthquakes around the world. Victoria, British Columbia is a community with substantial seismic hazard that has yet to experience a damaging earthquake and, thus, has not seen widespread interest in mitigating seismic risks. It is also a city with a substantial stock of URM buildings, constructed around the turn of the 20th century, reasonably similar in form to those devastated by the 2010/2011 Canterbury earthquake sequence in New Zealand.

To promote seismic upgrading of URM buildings in the region, cost-benefit analyses were undertaken specifically for Victoria, considering the seismic hazard, typical pedestrian and occupant exposure, building vulnerability, and local retrofit costs. The loss estimates are underpinned by motion-damage relationships derived in this thesis from observed damage in past earthquakes in California and New Zealand. Upgrading measures considered range from parapet bracing to full seismic upgrades consistent with local practices. Parapet bracing is shown to have favorable cost-benefit ratios – up to 4:1 for buildings on soft soils, indicating that the expected benefits outweigh the costs by a factor of four. Partial retrofits (eg. tying walls back to all floors/roofs, plus parapet bracing) are shown to have favourable cost/benefit ratios for buildings on soft soils (up to 1.7:1). Full upgrades are shown to have unfavorable cost-benefit ratios (maximum 0.7:1). In all cases, public benefits (i.e. reduced casualties) represent a significant portion of the total benefits, which provides evidence for cost sharing among building owners and the public.

The motion versus structural damage relationships derived in this study are also used in developing proposed refinements to the FEMA 154 screening methodology. New, additional score modifiers (specific to URM) are presented, including score modifiers for various levels of earthquake strengthening. A trial application on buildings in Victoria shows that the revised scoring system is able to more effectively differentiate amongst URM buildings, making it more suitable for URM-only surveys, such as are often implemented by communities interested in quantifying and mitigating URM seismic risk.

Preface

A significant portion of the material in Chapter 3 was published in a paper for the 12th Canadian Masonry Symposium: *Addressing URM Seismic Risk in Victoria Canada* (Paxton, et al. 2013). I was the lead author on this paper and was responsible for compiling the material on unreinforced masonry seismic risk mitigation in the United States. Co-author, Dr. Ken Elwood, prepared the material on URM seismic performance in the Canterbury earthquakes and co-author, Mr. Steve Barber, prepared material on the seismic retrofit incentives for heritage buildings in Victoria.

The damage statistics analyzed in Chapter 4 were provided to the author by others, who were responsible for performing the damage surveys and compiling the resulting databases. Databases for the Loma Prieta and Northridge earthquakes were graciously provided by Mr. Bret Lizundia of Rutherford + Chekene consulting engineers, who was responsible for collecting the data. A database for the Canterbury earthquakes was graciously provided to the author by Dr. Jason Ingham of The University of Auckland. Dr. Ingham, Dr. Michael Griffith of the University of Adelaide, and their team of researchers were responsible for collecting the data. Section 4.7 reviews the work completed by those responsible for compiling the databases, while Section 4.8 contains the new work, undertaken by the author.

The results and conclusions presented herein are the product of academic research and only reflect the views of the author. The results and conclusions do not necessarily reflect the views of UBC, the City of Victoria, or the Victoria Civic Heritage Trust. Any individuals using the results or conclusions presented herein do so at their own risk and the authors take no responsibility for such applications.

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I dedicate this thesis to my long-time girlfriend, Brandi Fredrickson, who without hesitation encouraged me to pursue graduate studies. Completing these studies has affected both of our lives and may this thesis be a reminder to me of how fortunate I am to have such a selfless and caring person with whom to share my life.

Chapter 1

Introduction

Unreinforced masonry (URM) was a common form of building construction in many cities around the turn of the 20th century throughout North America, and abroad. Despite their relatively well-known poor seismic behavior, URM buildings continue to be a major source of loss of life and property damage in earthquakes, and seismic risk mitigation programs are routinely met with substantial resistance. In virtually every case, risk mitigation measures were spurred on by emotional and political responses to losses in subject or nearby communities. The overarching purpose of this thesis is to provide rational evidence and decision-support tools promoting URM seismic risk mitigation and, thus, public safety. Particular focus is made on Victoria in application of the results obtained herein. However, the methodologies are applicable – and hopefully of interest – on a more general scale.

The following subsections provide some of the pertinent background information, define the scope and objectives of this study, and provide an outline of the general topics addressed herein.

1.1 Background

Where present, URM buildings affect nearly every member of a community, whether it is simply an office building one walks past, an apartment building one lives in, or a coffee shop one frequents. Moreover, seismic risk mitigation is a complex socioeconomic issue to which each community must find its own suitable solution. The following is a brief summary of the background information behind the initiation and execution of this study.

1.1.1 Project Partners & Structure

This study was funded through a Natural Sciences and Engineering Research Council (NSERC) Industrial Postgraduate Scholarship (IPS). Funding was jointly provided by NSERC and the “Industry Sponsor,” the Victoria Civic Heritage Trust (VCHT). A third partner to the study was the consulting engineering company, Read Jones Christoffersen Ltd. (RJC), with whom the author is employed. RJC provided office space for the author and both VCHT and RJC provided oversight and guidance for the

study. Finally, the study was supervised by Dr. Ken Elwood, Professor at the University of British Columbia in Vancouver.

Other parties also provided significant assistance throughout the process of this study, including undergraduate students at The University of British Columbia, the City of Victoria’s GIS department, researchers from New Zealand, and prominent engineers from California. Further details are provided in the Preface and Acknowledgement sections.

1.1.2 Limitations

The purpose of this study was not to identify or assess individual buildings in Victoria as vulnerable and, as such, the results are not fit for that purpose. Rather, the purpose was to develop methodologies that could be used to promote and implement URM seismic risk mitigation on a municipal basis in Victoria and in other interested communities.

The results and conclusions presented herein are the product of academic research and only reflect the views of the author. The results and conclusions do not necessarily reflect the views of UBC, the City of Victoria, or the Victoria Civic Heritage Trust. Any individuals using the results or conclusions presented herein do so at their own risk and the authors take no responsibility for such applications.

Individuals of varying expertise and experience were involved throughout the study and the results presented herein are based on a number of highly variable factors. The results are a general indication only and are not a replacement for a building-specific study by suitably qualified professionals.

1.1.3 About VCHT

The Victoria Civic Heritage Trust is a non-profit society in BC and a federally registered charity, whose mission statement is to *“work in co-operation with government and heritage agencies to develop, administer, and financially support programs that preserve, promote, interpret, and enhance the cultural and natural heritage resources of the City of Victoria and its environs.”* The VCHT was created in 1989 by the City of Victoria as a civic vehicle to administer heritage incentive programs and initiatives on behalf of the city. Through annual capital and operating funds from the City, the VCHT supports research, conservation, and rehabilitation efforts for legally protected commercial, industrial, and institutional heritage buildings in Victoria, through cash and tax incentives. The VCHT board of directors is comprised of professionals with specific

knowledge of heritage buildings and construction: developers, contractors, city planners, professional engineers, and architects.

More recently, VCHT has become interested in developing new incentive programs to address the need for partial upgrading of URM buildings (i.e. those that will not undergo full seismic upgrading due to a re-development or change of occupancy). VCHT's interest in initiating this study stemmed from the need to develop substantiating material on the benefits of partial seismic upgrades such as parapet bracing.

1.1.4 The Climate and Context of Seismic Risk and Mitigation Measures

Various communities around the world have made strides in mitigating URM seismic risk. However, many more have yet to make virtually any progress whatsoever.

As shown in Chapter 3, efforts in Victoria are lacking compared to many other jurisdictions facing similar seismic hazards. While certain parties are acutely aware of the risk, such as emergency managers and heritage planners, there has been no political movement to speak of in terms of seismic risk mitigation. In general, the problem has yet to be openly and specifically addressed on the public stage.

This is in stark contrast to areas in California where seismic risk mitigation has been an important political topic for decades (and continues to be one). More recently, efforts in the pacific northwest of the USA have started to develop, including the City of Seattle and the state of Oregon (see Chapter 3).

1.1.5 URM Seismic Risk in Victoria

Victoria lies in the Cascadia Subduction Zone (CSZ), which is a somewhat special seismic setting. Figure 1.1 shows the CSZ. Many areas, such as California, are susceptible to one main type of earthquake, known as “crustal” earthquakes. In the CSZ, there are two more potential types of earthquakes: “subcrustal” and “subduction” earthquakes. The two former types tend to be small to moderate in size, but happen more frequently (and possibly close to any given area). The latter is much larger (they are sometimes referred to as “mega-thrust” earthquakes); they occur much less frequently, but affect larger areas with high intensity, long duration shaking.

Crustal and Subcrustal earthquakes are more commonly observed around the world. An example of a crustal earthquake is the 2011 Christchurch earthquake (in New Zealand) and an example of a subcrustal earthquake is the 2001 Nisqually earthquake in

Washington State. An example of a subduction earthquake is the 2011 Tohoku earthquake in Japan.

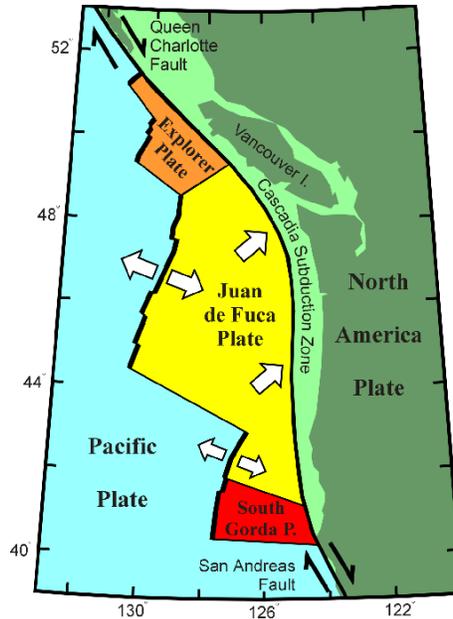


Figure 1.1 – The Cascadian Subduction Zone

From: http://commons.bcit.ca/civil/students/earthquakes/unit1_02.htm

Victoria has not experienced a damaging earthquake in its history. However, this is merely good fortune. Potentially damaging earthquakes have occurred all around Victoria over the past 100 years, and Victoria is considered to have one of the highest seismic hazards in Canada (NRC 2010). Figure 1.2 shows some significant earthquakes that have occurred in the general region.

To put matters in statistical terms, consider the following earthquake probabilities (Onur and Seeman 2004):

- The probability of a “structurally damaging” ($MMI \geq 7$) crustal or subcrustal earthquake for Victoria in the next 50 years is 21%
- The probability of a “non-structurally damaging” ($MMI \geq 6$) crustal or subcrustal earthquake for Victoria in the next 50 years is 53%
- The probability of a M9 mega-thrust earthquake (which is expected to produce $MMI=7$ shaking in Victoria as per USGS, 2011a) in the next 50 years is 11%

Note that “non-structurally damaging” refers to a level of shaking at which most existing structures would have damage to only “non-structural” items (eg. partitions, windows, ceiling, contents). As will be seen in this study, however, unstrengthened unreinforced masonry buildings will likely experience some structural damage at this level.

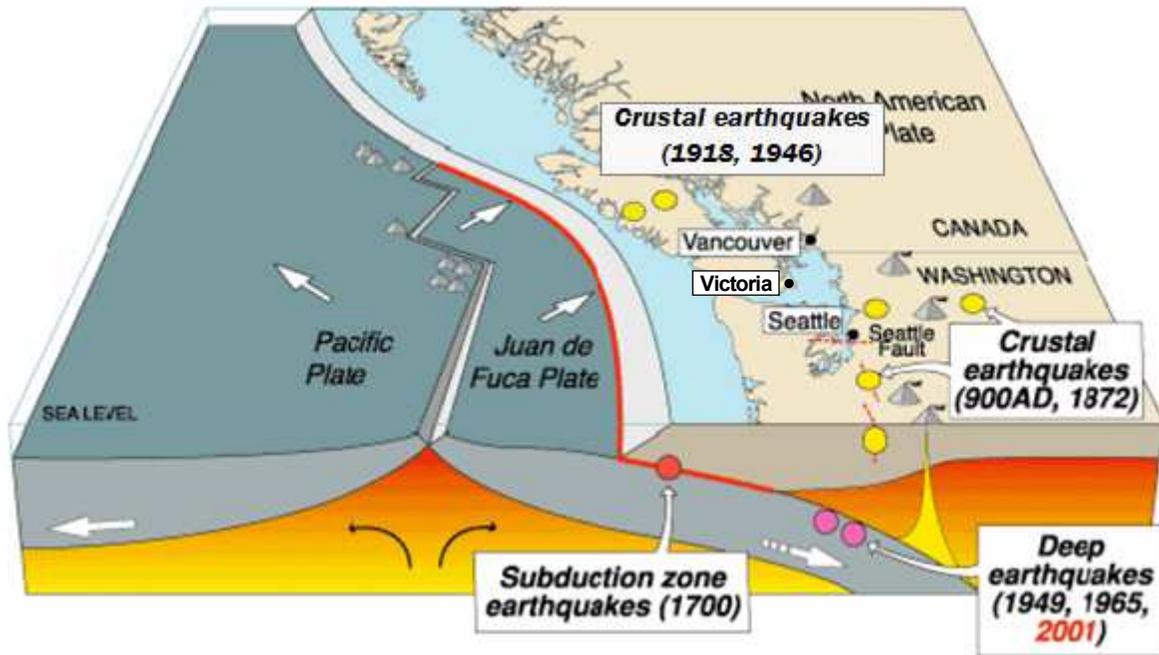


Figure 1.2 – Seismic History of The Cascadia Region

Source: <http://www.quaketrip.com/wp-content/uploads/2011/01/CascadiaSubductionZone2.jpg>

1.2 Purpose

The author wishes to note that this thesis is perhaps different from most others in that it has two primary purposes: as with every other academic thesis, this study aims to contribute new knowledge to its field; additionally, it also has the purpose of gathering existing information for practical uses by the industry sponsor. Both purposes, however, are highly valuable in improving public safety.

1.3 Scope

This study was one that touched on many different aspects, some of which fell beyond the author’s specific field of expertise. Any one aspect (for example, socioeconomic impacts of retrofit ordinances, seismic motion-damage relationships, or the monetary value of a statistical life) could easily merit a much more detailed treatment. Although isolating one element and perfecting it is valuable in academia, such an effort would not

have been nearly as useful to the industry sponsor. In general, therefore, efforts were only concentrated on a given aspect to a point where it was felt that further efforts would not significantly change the conclusions reached in this specific study. However, a careful effort has been made to document the rationale and decisions so that future researchers may readily understand them and modify/improve upon the work contained herein.

There are many types of unreinforced masonry buildings around the world. This thesis focuses on clay brick URM bearing wall buildings that were typically constructed in the late 19th and early 20th centuries on the west coast of North America. Ultimately, we are interested in the performance of Victoria’s buildings, which are quite consistent with those in many other locations on the west coast of North America. URM buildings in New Zealand are also studied and prove to be somewhat similar.

1.4 Impetus & Objectives for The Study

Unreinforced masonry buildings have proven to be a leading source of loss of life and property damage again and again in earthquakes around the world. Unfortunately, the high economic and social costs of seismic rehabilitation¹ (and associated public opposition) frequently hamstring risk mitigation efforts.

The 2010/2011 Canterbury (New Zealand) earthquakes have once again raised awareness regarding the seismic vulnerability of unreinforced masonry buildings. The highly-publicized losses from the February 2011 Christchurch earthquake included:

- 39 deaths attributed to URM buildings (Canterbury Earthquakes Royal Commission 2012)
- The partial or full demolition of 242 of 252 heritage buildings within the city, most of which were of URM construction (CERA 2014). While damage was severe, heritage agencies reported that the swift demolition process eliminated more buildings than was necessary (The Press 2011)
- The widespread closure of the city’s central business district, a significant portion of which remained closed for over one year as officials assessed and demolished unsafe buildings, most of which were of URM construction

¹ Throughout this this thesis, the terms “upgrading,” “strengthening,” “rehabilitation,” and “retrofitting” are variously used, all of which are found in the literature and in industry to some degree. For the purposes of this thesis, all of the above terms can be taken as synonyms.

These losses show the need for effective risk mitigation strategies and this lesson has reverberated strongly among many concerned parties in Victoria, BC. Victoria is markedly similar to Christchurch in many ways, including British colonial history, historic masonry construction, population size and demographics, importance of tourism to the economy and, most importantly, seismic hazard.

This study was initiated by the industry sponsor, the Victoria Civic Heritage Trust as part of a multi-pronged approach to URM seismic risk mitigation, including retrofit incentives, public outreach, and research. The high-level objectives identified for this study were as follows:

- 1) To gain an improved understanding of the seismic risk due to unreinforced masonry buildings in “Old Town” (an area of Victoria’s downtown core)
- 2) To develop material for educating stakeholders about the risks
- 3) To develop a rational and scientific basis for future work by VCHT in encouraging seismic upgrading through existing and new incentive programs
- 4) To develop a rational and scientific basis for future work by city officials in developing a URM seismic risk mitigation program
- 5) To develop methodologies that can be extended to other communities

These objectives will be revisited in the conclusions section (Chapter 8) to discuss how they were fulfilled by this study.

1.5 Organization of Thesis

Because the purpose of this thesis is to fulfill several needs of the industry sponsor and because unreinforced masonry buildings pose a complex socioeconomic problem, this thesis is quite broad in nature. Some chapters are intended to educate those unfamiliar with URM buildings and how their risks are typically mitigated, while other chapters are highly detailed and technical in quantifying the seismic performance of URM buildings and the benefits of strengthening. The outline for this thesis is as follows:

Chapter 2 – Characterizing URM Bearing Wall Buildings: defines the typical buildings that are the subject of this study, and their components. Typical construction practices are discussed, as are the common deficiencies and strengthening measures.

Chapter 3 – URM Seismic Risk Mitigation Programs: reviews the efforts in several regions throughout North America and abroad and compares them to the efforts

to date in Victoria (and southwestern BC in general). Retrofit ordinances, incentives, and financing schemes are discussed.

Chapter 4 – Quantifying Building Vulnerability: past earthquakes in California and New Zealand have provided valuable insight into the seismic performance of both strengthened and unstrengthened buildings. To quantify the performance in an objective manner, we turn to statistical analyses. This chapter reviews the available statistical data on seismic damage to URM buildings and makes use of previously collected raw data to generate new results, specifically for the purpose of this study. The final product is motion-structural damage relationships for URM buildings of various strengthening levels (including partial strengthening, such as parapet bracing).

Chapter 5 – Cost-Benefit Analysis for URM Seismic Rehabilitation: making use of the motion-structural damage relationships derived in Chapter 4, other existing relationships for non-structural components, cost analyses for seismic strengthening, and many other inputs, a cost-benefit analysis is performed for seismic strengthening of a prototypical URM building in Victoria and a sensitivity analysis is conducted. Strengthening levels considered include parapet bracing, tension ties for all floors, and comprehensive seismic rehabilitation.

Chapter 6 – Assessing and Prioritizing URM Seismic Risk: reviews several different seismic screening methodologies in the literature. Merits and flaws specific to the assessment of URM buildings are discussed. Ultimately, modifications to an existing procedure (FEMA 154) are provided to render the assessment better suited to the subject URM buildings.

Chapter 7 – Inventory and Screening of Victoria’s URM Buildings: reviews the available data on URM buildings in Victoria and presents a variety of summary statistics, characterizing the building stock. The need for the City of Victoria to commission a new, complete inventory of its URM buildings is highlighted and a potential methodology (including the newly modified assessment procedure) is presented. Summary statistics for a pilot survey of 81 buildings in Victoria are presented.

Chapter 8 – Summary and Conclusions: aims to succinctly summarize the results of the study.

Chapter 2

Characterizing URM Bearing Wall Buildings

2.1 Purpose and Scope

In this chapter, the buildings that will be the focus of our study are defined. This will include an introduction for those unfamiliar with URM² buildings, a discussion on their seismic behavior, a review of the commonly observed failure modes, and common measures for seismic rehabilitation.

2.2 Defining Typical Unreinforced Masonry Buildings

URM buildings are present around the world and come in a great variety of forms. In this study, however, the focus is primarily on the type of URM building that is most typical on the west coast of North America. The word ‘typical’ is used loosely here to indicate that the structural materials and assemblies are most commonly observed. This is an effective approach because there is relatively little variation in URM construction in this region, as it was sparsely populated before the early to mid-19th century (thus what small structures existed were built of wood) and URM construction fell out of favour in the early to mid-20th century as its seismic vulnerability became apparent.

Figure 2.1 shows some common components of a URM building. It should be noted that the building shown in the figure is not itself “typical” – it is rather large and complex so as to include all the possible components (and in some cases, alternate components, such as components #4 and 5). As will be shown later in this chapter, smaller two and three storey buildings, often constructed in rows immediately adjacent one another, are more common in many areas, including Victoria. The essential characteristics of a typical west coast URM building are:

- Bearing walls constructed from clay bricks and lime-based mortars, composed of multiple wythes
- Stone rubble foundations

² URM buildings other than clay brick bearing wall buildings are rare in Victoria and this study focuses essentially solely on clay brick bearing wall buildings. Throughout this thesis, the term “URM” refers specifically to clay brick URM bearing wall buildings, unless otherwise noted.

- Timber floor/roof structures, composed of board sheathing and rough sawn lumber joists (occasionally supported on heavy timber interior framing)
- Open fronts (i.e. little to no wall) are common for row buildings

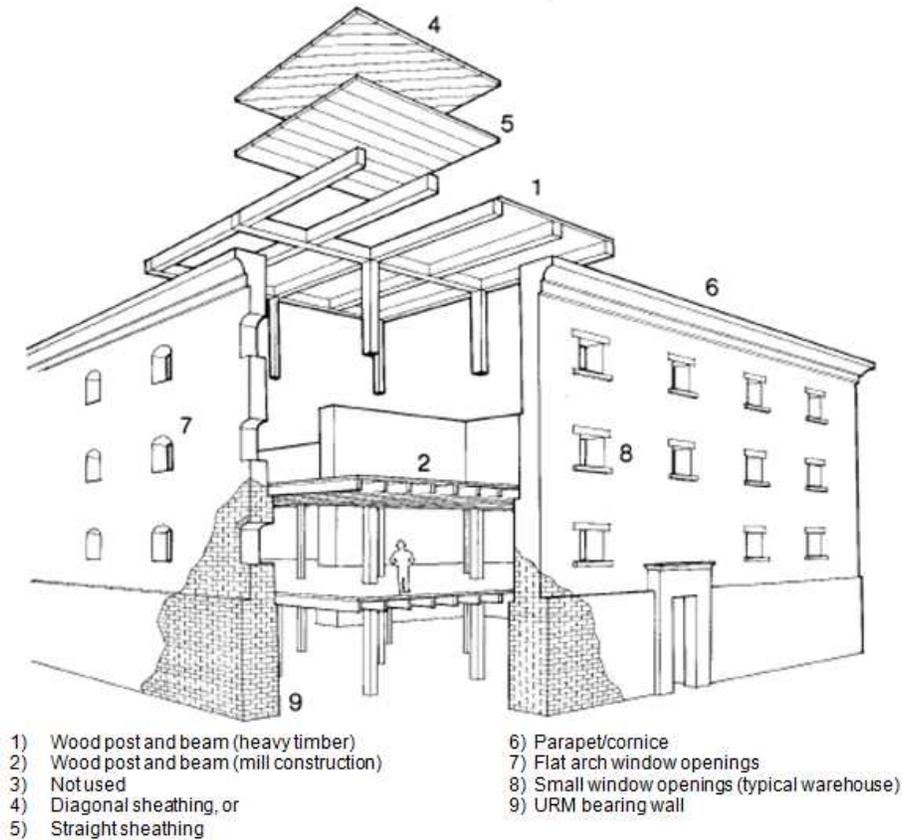


Figure 2.1 – Typical URM Building and Components

Modified From: FEMA, 2002a

Based on the literature (ABK 1981, Lizundia, Dong and Holmes 1993, Rutherford & Chekene 1997) and discussion with practicing engineers and researchers, it is understood that this type of URM construction is common throughout western North America. Of course, there are many variations in structural form fitting this broad description and the structural form of a URM building is often quite indicative of its original use. Rutherford and Chekene (1990) defined 15 prototype URM buildings for San Francisco, U.S.A. based largely on original use. The illustration from the document is reproduced here, as Figure 2.2. This essentially shows the variety of URM buildings that is commonly encountered on the west coast of North America, including Victoria. All would fit the aforementioned broad description. Figure 2.3 shows photos of a few example buildings in Victoria.

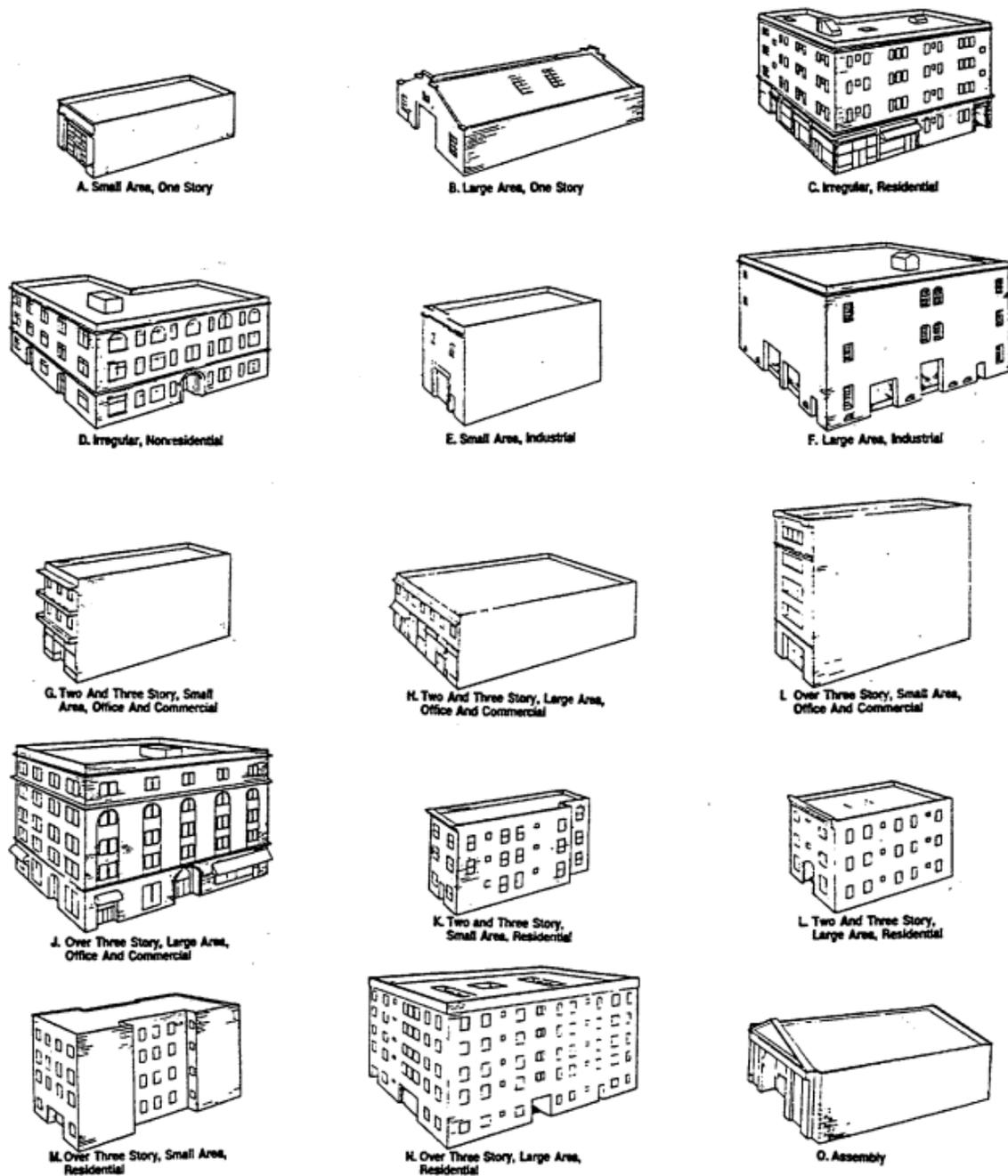


Figure 2.2 – San Francisco URM Prototypes

From: Rutherford and Chekene, 1990

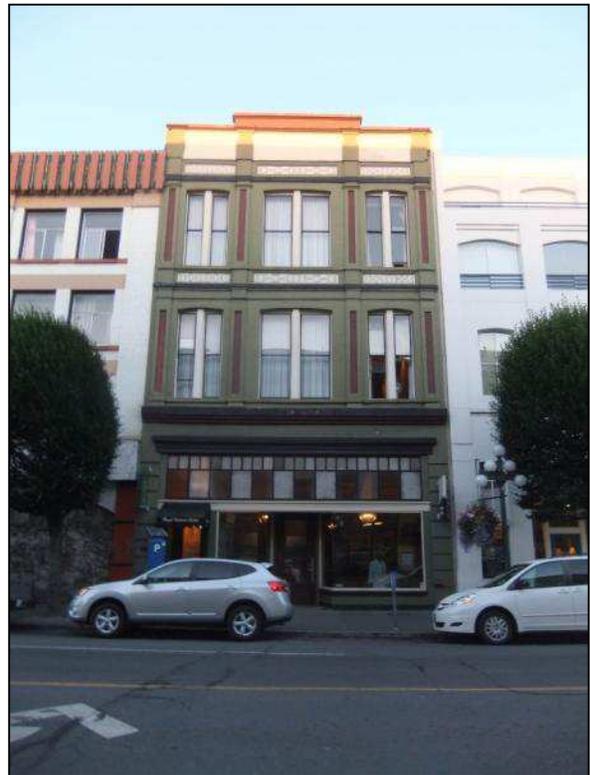
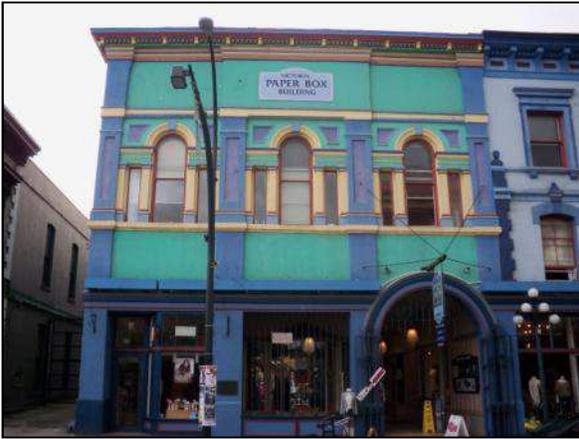


Figure 2.3 – Examples of URM Buildings in Victoria, BC

2.3 Structural Model

While seemingly simple in form, the seismic behavior of URM buildings is substantially different from typical modern buildings and, perhaps, more complex.

In modern buildings constructed of concrete or steel, a greater proportion of the mass of the structure is located at the floor levels, due to the presence of floors slabs. These floor slabs, which act as diaphragms, often possess much more flexural stiffness than do the vertical elements of the seismic force resisting elements (eg. moment frames, shear walls, braced frames). As such, it is typical in seismic design and analysis of new buildings to represent the dynamic behavior of the building with a “lumped mass” model, in which all dynamic response amplification is assumed to occur in the vertical components of the SFRS. The diaphragms are assumed to be rigid and thus experience no further dynamic amplifications. This is the basis of the seismic design provisions for new buildings in modern codes such as the National Building Code of Canada (NRC 2010), the International Building Code (ICC 2012) and the New Zealand Standard NZS 1170.5-2004 (NZS 2004).

Many URM buildings, however, are not accurately represented by this model. They tend to have very light and flexible wood diaphragms, typically consisting of plank sheathing (which is even more flexible than modern blocked plywood diaphragms) and the exterior walls are quite heavy and stiff. The result is that URM buildings are essentially the opposite of most modern buildings in terms of their distribution of mass and stiffness: the “side walls” (also known as “end walls” as per Figure 2.4) are often assumed to be extremely stiff and, thus, experience negligible dynamic amplification; the diaphragms are more flexible and experience significant dynamic response amplification.

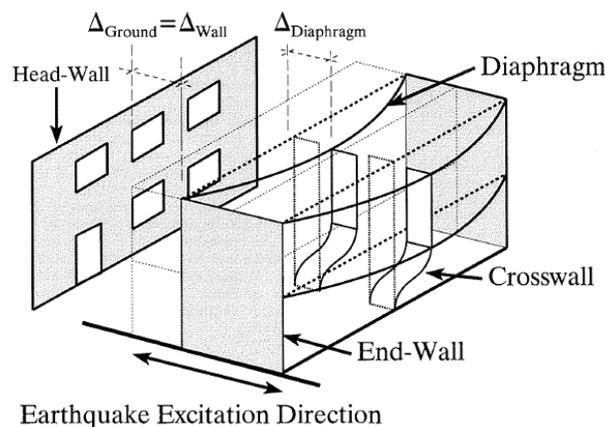


Figure 2.4 – URM Building Seismic Load Path

(From Bruneau, 1994)

This is the basis of many URM seismic assessment procedures, such as *Canada’s Seismic Evaluation Guidelines for Existing Buildings* (NRC 1992) and the *International Existing Building Code* (ICC 2012a). Figure 2.5 and Figure 2.6 below illustrate the two different structural models.

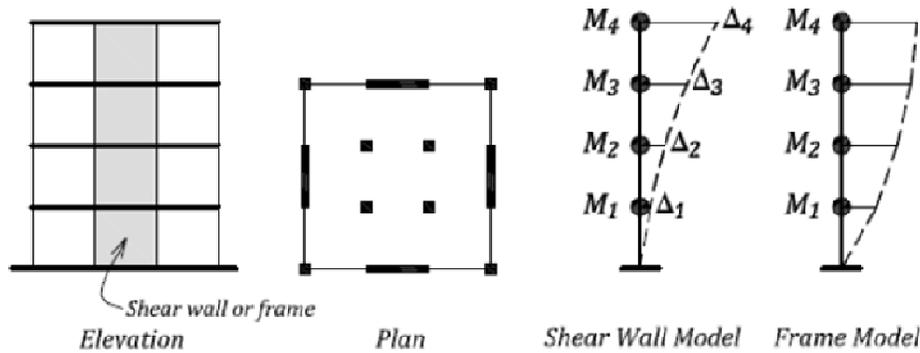


Figure 2.5 – Dynamic Model with Rigid Diaphragm & Flexible Walls
From: CCMPA, 2009

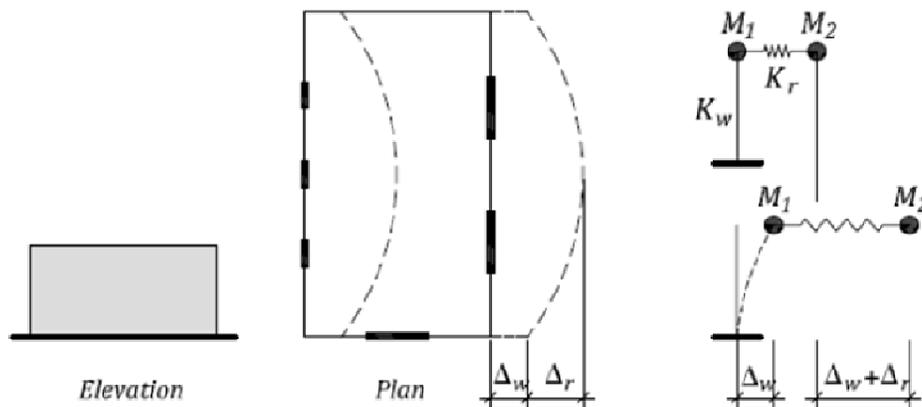


Figure 2.6 – Dynamic Models with Flexible Diaphragm (Walls May be Rigid)
From: CCMPA, 2009

Provided sufficient connections exist, the seismic load path is idealized as such:

- 1) The ground motion excites the URM end walls
- 2) The end walls transmit the motion to the wood diaphragms
- 3) The diaphragms transmit the motion (and provide restraint) to the head walls
- 4) The head walls respond dynamically in their out-of-plane direction

Many unretrofitted buildings lack sufficient connection at one or more points in the seismic load path and this is the greatest seismic vulnerability of URM buildings. Several post-earthquake reconnaissance reports have cited a lack of sufficient connection between diaphragms and walls as a common reason for damage and collapses (Deppe 1988, Bruneau 1990, LATF 1994, Ingham and Griffith 2011b).

2.4 Components of a Typical URM Building

Having defined the form of a typical URM building and formulated a structural model, some of the more common components will now be examined.

2.4.1 Unreinforced Masonry Walls

URM walls are composed of some form of unit block (eg. clay bricks, stones, concrete block) and, typically, a mortar (although dry stacked masonry also exists). In this study, we restrict our focus to URM buildings with walls constructed of clay bricks and mortar. Bricks used in building construction were typically fired at local plants and mortar composition was either sand-lime or a blend of lime and portland cement with sand, dependent upon the era of construction: before the turn of the 20th century, sand-lime mortars were used almost exclusively for URM building construction (ASTM 2003). Note that the primary difference between sand-lime and sand-lime-cement mortars is that the latter cures quicker and achieves higher strengths. While the higher strength may be desirable from a seismic point of view, there are a variety of reasons why repointing (a term for replacing the mortar) should use a similar mortar to that of the original construction. Among these, is the fact that a strong, stiffer mortar may attract too much stress which could damage the adjacent masonry (ASTM 2003); such mortars also tend to be less permeable, which could lead to increased water transmission through the bricks, which is undesirable.

The walls are constructed by laying the bricks atop one another in a bed of mortar. Each vertical layer of bricks is called a “course” and the mortar between the courses is called the “bed joint.” Mortar is also applied between the ends of the bricks in what is called a “head joint.” Structural walls are generally several layers (also known as “wythes”) wide. Between these wythes, is another mortar joint called a “collar joint.” Figure 2.7 shows courses and wythes within a wall.

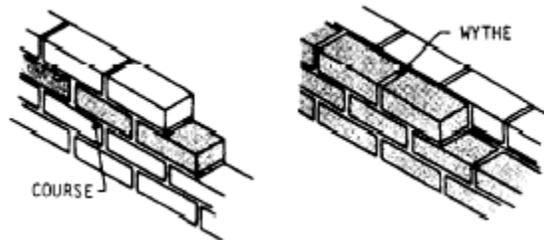


Figure 2.7 – Courses and Wythes of Unreinforced Masonry

From: <http://www.tpub.com/engbas/7-32.htm>

The bricks can be laid in various orientations. The typical orientation (shown in Figure 2.7) is known as a “stretcher” orientation. In a multi-wythe wall, both wythes are laid primarily in the stretcher orientation and the wythes are tied together by laying a course of bricks in the “header” position every five or six courses. Figure 2.8 shows stretcher and header courses.

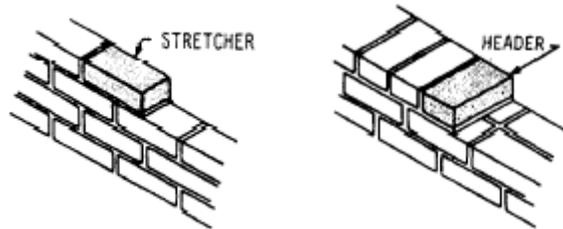


Figure 2.8 – Stretcher and Header Courses of Unreinforced Masonry

From: <http://www.tpub.com/engbas/7-32.htm>

Walls may also have an additional wythe that is not tied in with header courses and may have an air gap. This is known as a “veneer wythe” – they are considered non-structural and are only connected to the structural wall by metal ties at some regular spacing. In modern buildings, the ties are designed to transfer the seismic loads of the veneer back to the structural wall. In URM buildings, however, the ties are generally quite minimal and may be severely deteriorated due to corrosion; thus, they are often quite vulnerable to seismic damage and pose a significant fall hazard. Veneer wythes can be identified by the lack of header courses. For URM buildings, veneer wythes are typically an architectural feature that is only present on main facades (if at all). Figure 2.9 shows a photo of a building with and without veneer wythes as well as a photo of veneer ties exposed at an area of excavated bricks. Note the absence of header courses in the veneer wythe.

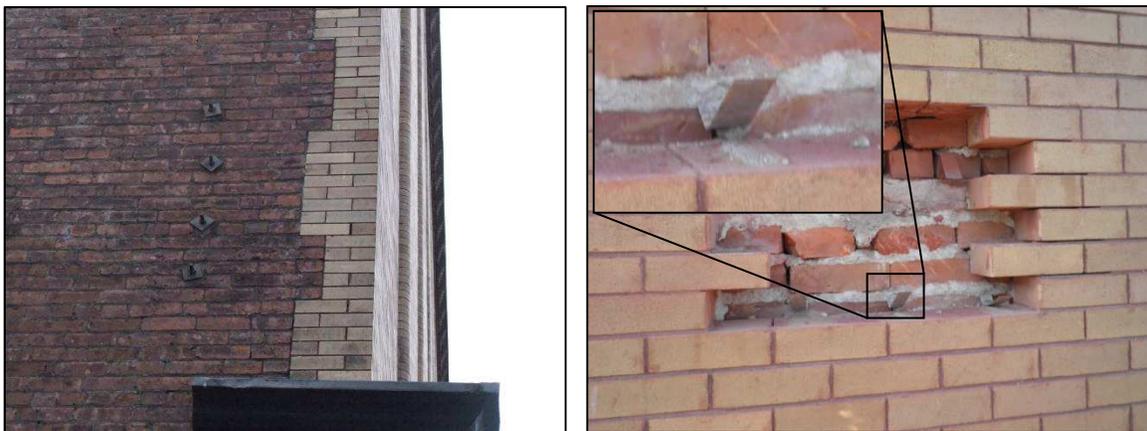


Figure 2.9 – URM Building with a Veneer Wythe (Left) and Veneer Ties (Right)

Photo credit: Read Jones Christoffersen Ltd.

2.4.2 Interior Framing

Wood is typically the material of choice for interior framing in URM buildings. Depending on the size and original use of the building, this may include light framing such as studs and joists (similar to a modern building), or it may include large, heavy timber post and beam type construction. The latter is typically found in larger structures that originally served as warehouses or some other industrial purpose. Occasionally, interior framing will also consist of structural steel or reinforced concrete beams and/or columns. Figure 2.10 shows various interior framing elements. Of course, interior walls constructed of URM or other archaic materials are possible.



Figure 2.10 – Light Wood (Top Left), Heavy Timber (Top Right), Structural Steel (Bottom Left), and Reinforced Concrete (Bottom Right) Framing

Photo credit: Read Jones Christoffersen Ltd.

2.4.3 Floor and Roof Diaphragms

Wood framing is by far the most common type of floor and roof construction for URM buildings on the west coast of North America. It is acknowledged that other types are occasionally encountered, but this study will focus almost exclusively on flexible wood diaphragm type URM buildings.

Wood floor and roof structures in URM buildings are typically comprised of some form of sheathing material atop rough sawn wood joists, spaced at 12”-24” on centre. Original sheathing material is often flat boards, about 6” wide and ½” to 1” thick, laid with small gaps between each board. The boards may be laid perpendicular to the joists (“straight sheathing”), or at an angle (“diagonal sheathing”) and are typically fastened with two nails at every second or third joist. The orientation is significant because it affects the strength and stiffness of the diaphragm. Occasionally, two layers of sheathing may be present. Joist size varies depending on the span, typically falling between 2”x6” and 3”x12.” Panel sheathing such as plywood may be encountered, typically installed as part of past renovations. In buildings with heavy timber framing, floors may be constructed of rough sawn lumber laid immediately next to one another to form a solid surface. Figure 2.11 provides photos of various floor construction types.

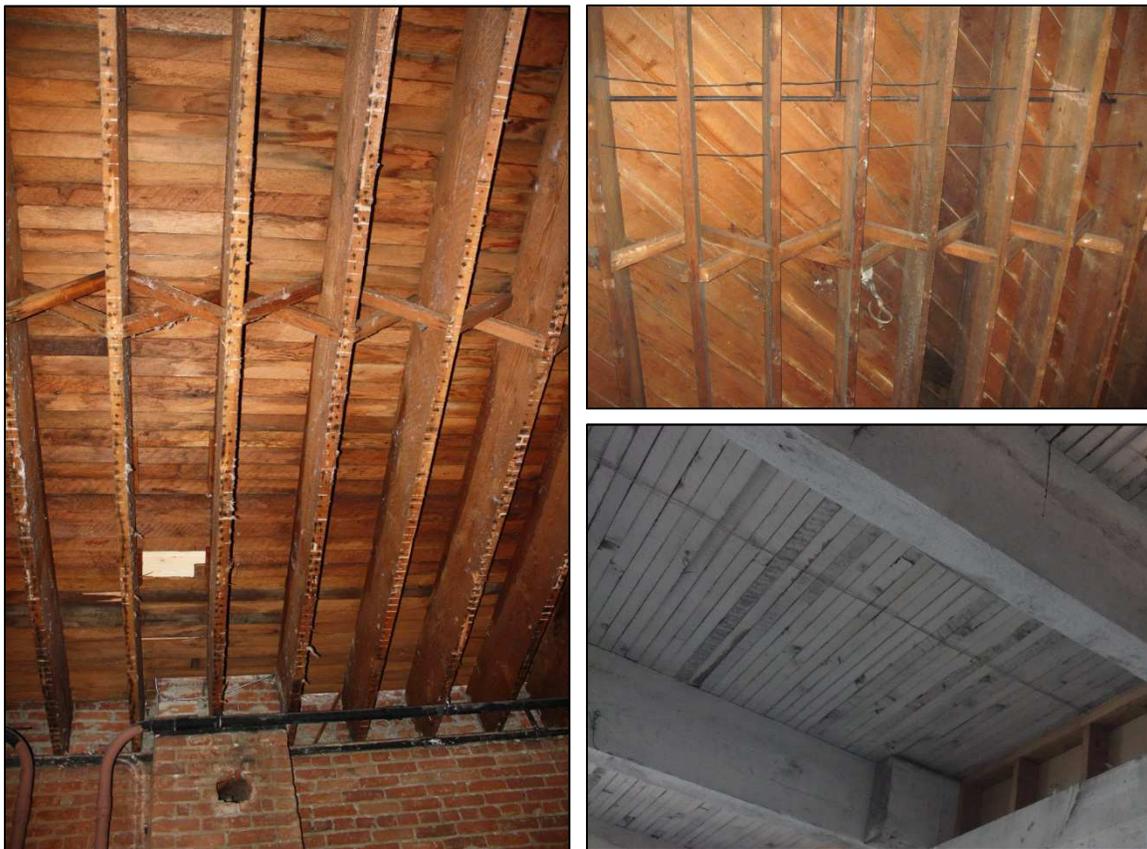


Figure 2.11 – Straight Sheathing (Left), Diagonal Sheathing (Top Right), Solid Laminated Dimensional Lumber (Bottom Right)

Photo credit: Read Jones Christoffersen Ltd.

2.4.4 Wall to Floor/Roof Connection

Another key component of diaphragms is their connection to the URM walls. The two most common conditions are for joists/beams to sit on ledges (created by reducing the wall thickness by one wythe) or in pockets formed into the wall lay-up. In both cases, the floor framing may or may not be anchored to the wall to resist lateral loading (eg. from wind and earthquake) as part of the original construction; the frequency of anchorage varies by region, presumably due to local construction practices of the day. In any case, the anchorage (if present at all) is generally not capable of resisting the loads imposed by any modern seismic codes. Figures 2.12a shows ledge and pocket details and Figure 2.12b shows ties from original construction.



Figure 2.12a – Floor Framing at a Ledge (Left) and a Pocket (Right)



Figure 2.12b – Floor-to-Wall Anchors Viewed from Inside (Left) & Outside (Right)

Photo credit: Read Jones Christoffersen Ltd.

2.4.5 Building Appurtenances

Common building appurtenances of interest for URM buildings include parapets, cornices, corbels, and chimneys. For the purposes of this study, we are essentially interested in any component that poses a fall hazard. The following sections discuss the individual components.

2.4.5.1 Parapets

Parapets are the portions of URM exterior walls that extend above the roof. They serve many functions, but were originally provided to prevent fire from spreading from rooftop to rooftop (Rutherford & Chekene 1997). As URM construction became more popular in the early 1900's, parapets became an important architectural feature and character defining element of a building, and often were often large and ornately decorated, particularly on the main facades. Height varies from just a few inches above the roof surface to several feet and the parapet may project straight up from the wall below or be corbelled outwards in a decorative fashion. Occasionally, concrete parapets will be found on URM buildings. Figure 2.13 shows examples photos of parapets.



Figure 2.13 – Masonry Parapets Viewed From Street (Top Left) and Rooftop (Top Right); Corbelled Parapet (Bottom Left); Concrete Parapet (Bottom Right)

2.4.5.2 Cornices

Cornices are often constructed from lightweight materials such as wood and sheet metal, although terra cotta (which is heavy) is also common. Lightweight cornices obviously pose a significantly lower fall hazard, but the consequences of a large, heavy cornice falling to the streets can be similar to those of a parapet. Figure 2.14 provides example photos of cornices.

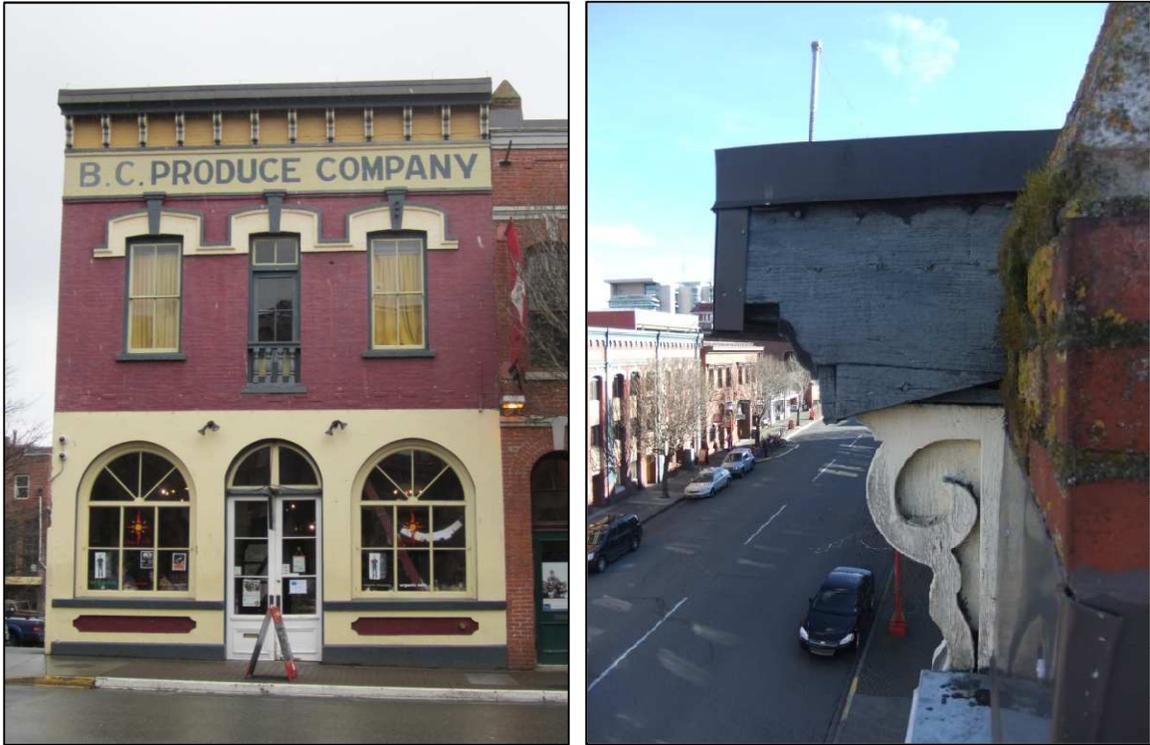


Figure 2.14 – Example Photos of a Typical Lightweight Cornice

2.4.5.3 Chimneys/Pilasters

Chimneys and pilasters are common elements on URM buildings that are constructed of URM and pose significant fall hazards to life-safety. Because all four sides are exposed to weather, there is often a greater fraction of the mortar in a deteriorated condition. For chimneys, the portion projecting from the roof may be just one wythe thick and the result is that they are highly vulnerable to earthquakes. Pilasters may appear similar to chimneys, but are generally solid sections. It should also be noted that these elements are not always located at the perimeter of a building and, thus, may not be readily visible from street level. Figure 2.15 provides example photos of chimneys and pilasters.



Figure 2.15 – Example Photos of Chimneys & Pilasters

2.4.5.4 Gables

Buildings with pitched roofs often employ gable ends, resulting in an increased wall height. There is also typically little dead load or restraint from the roof, as the roof framing spans parallel to the gable. As a result, gables are also highly vulnerable elements. Although considered part of the structural wall (and thus not a true appurtenance), it is identified in this section because it poses a fall hazard similar to the other elements herein. Figure 2.16 shows examples photos of gable ends.



Figure 2.16 – Example Photos of Gables

2.5 Failure Modes

Because URM buildings were constructed long before the implementation of seismic design provisions and, in some cases, building codes whatsoever, there are several possible failure modes under seismic loading that were not addressed in their design and construction. The following sections present the commonly observed failure modes. It draws primarily on the work of others (Rutherford & Chekene 1990, Ingham and Griffith 2011a, Ingham and Griffith 2011b) and is included as part of the practical information

gathering for the industry sponsor for the purposes of educating various stakeholders about the risk. Also, because the primary purpose of this section is to educate non-technical personnel, discussion on the failure modes is quite limited. Readers interested in a more technical analysis should refer to the original documents and related literature.

2.5.1 Parapet, Gable, and Chimney Failure

Parapets, gables, and chimneys are widely accepted to be among the most vulnerable components of URM buildings (Deppe 1988, Bruneau 1990, Lizundia, Dong and Holmes 1993, Rutherford & Chekene 1997). In the case of parapet failure, the wall projecting passed the roof is in a cantilever configuration and is subject to dynamic loading from its own inertia and the diaphragm. If the flexural load on the cantilever results in a stress that exceeds the tensile capacity of the masonry, a crack will occur and the parapet will begin to rock. Depending on the remainder of the excitation, the parapet may topple over or remain dynamically stable and eventually come to rest. The failure of gables and chimneys is similar. All such failures pose life-safety threats and may also block roads and sidewalks, thereby hampering rescue efforts. Figure 2.17 provides photos of parapet, gable, and chimney failures.



Figure 2.17 – Failure of Parapets (Left), Gables (Right) and Chimneys (Bottom)

From: Ingham & Griffith, 2011a

2.5.2 Cornice Failures

Cornices fail due to insufficient anchorage to the building face, either because of the original construction or deterioration of the masonry at the anchorage. Many cornices are constructed of lightweight material, and thus the seismic demands are reduced and the life-safety threat is diminished. Heavier cornices pose hazards similar to parapets.

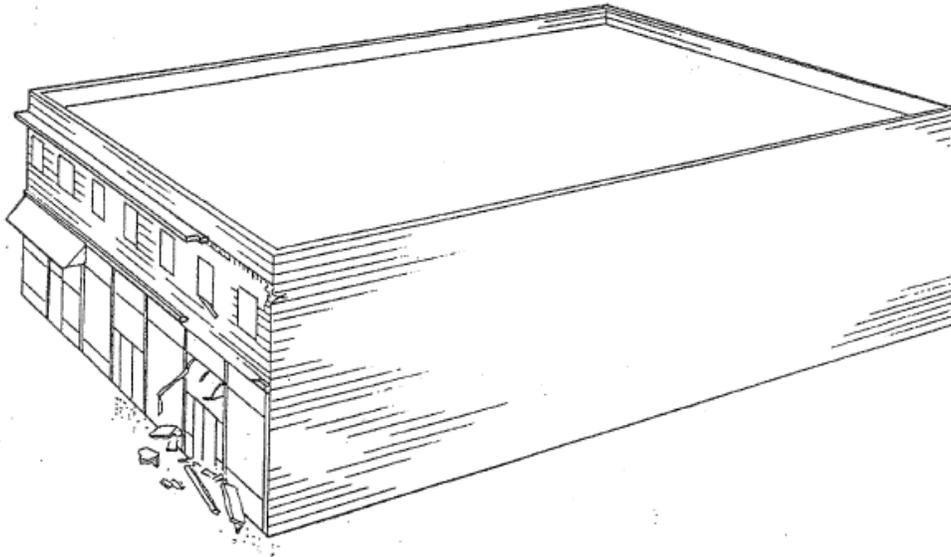


Figure 2.18 – Cornice Failure

From: Rutherford and Chekene, 1990

2.5.3 Veneer Failure

As mentioned in Section 2.4.1, anchorage of veneers to the structural backing wythes in URM buildings often consists of either corrugated metal ties, long nails, or other miscellaneous steel ties. The seismic resistance of the ties is further reduced due to low strength, high-lime content mortar typically used in URM buildings. Figure 2.19 provides a photo of a veneer failure.



Figure 2.19 – Veneer Failure

From: Ingham and Griffith, 2011b

2.5.4 Out-of-Plane Wall Failure

Under shaking of increased intensity or duration, entire walls may topple outwards. This is due to either a lack of anchorage or an excessive flexural stress in the wall as it bends between floors. The life-safety hazard to pedestrians is increased due to the increased volume of debris falling outwards. Often, the interior walls and columns are sufficient to support the floors and the remainder of the structure is left standing, while the wall lies on the ground in ruins. As pointed out by Ingham and Griffith (2011b), the hazard is greater to the public on the street than the occupants of building. Figures 2.20 and 2.21 provide an illustration and a photo of an out-of-plane (OoP) failure.

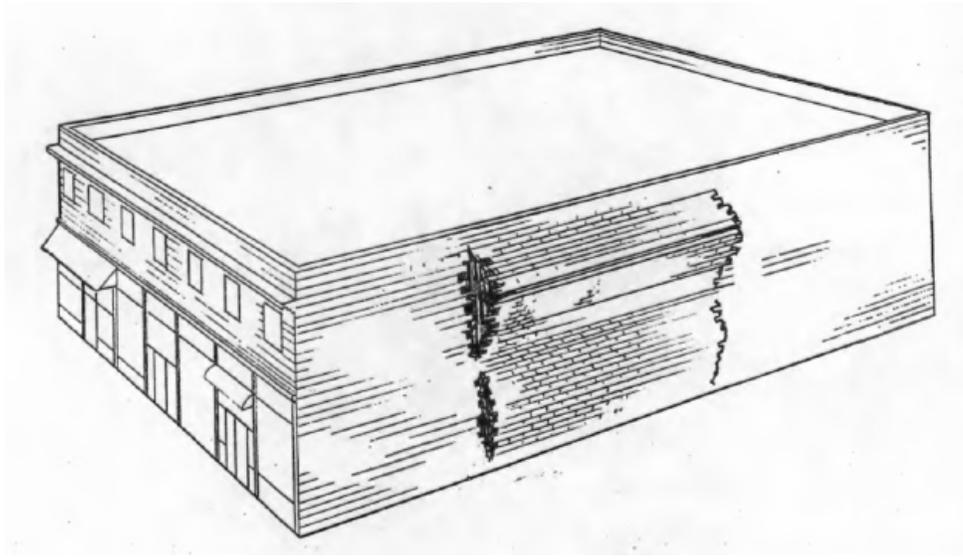


Figure 2.20 – Illustration of Out-of-Plane Wall Failure

From: Rutherford and Chekene, 1990



Figure 2.21 – Photo of Out-of-Plane Wall Failure

From: Ingham and Griffith, 2011b

2.5.5 In-Plane Wall Failure

Significant damage can also result from in-plane seismic actions. The most commonly observed in-plane failure is an ‘X’ crack shear failure; other in-plane failures include bed joint sliding shear and toe crushing. In general, in-plane failures pose less of a life safety threat. Figures 2.22 and 2.23 provide an illustration and a photo, respectively. Note that although the life-safety threat is considered lower for such failures, it is still significant, especially if the damage is widespread. The building shown in the photo below was ultimately demolished due to safety concerns.

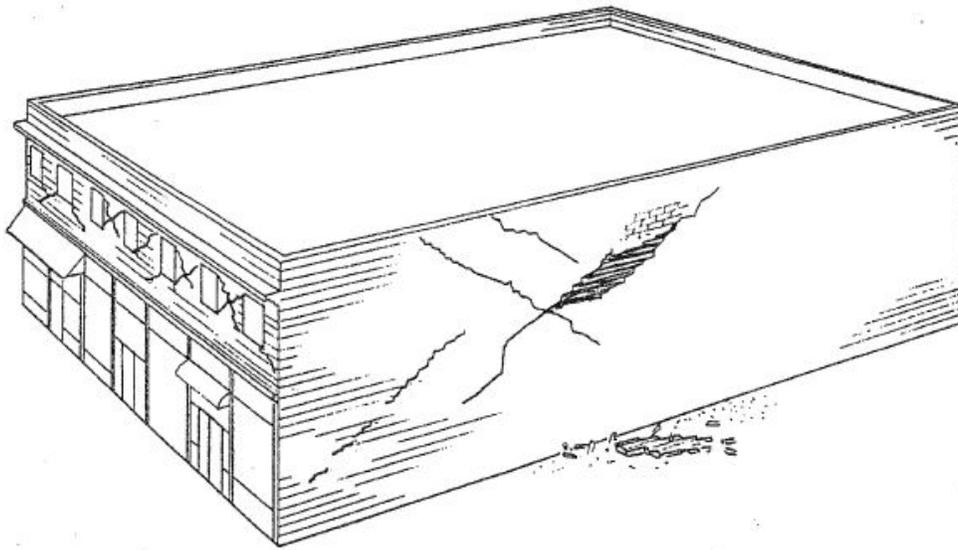


Figure 2.22 – Illustration of In-Plane Shear Failure

From: Rutherford and Chekene, 1990



Figure 2.23 – Photo of In-Plane Shear Failure

From: Ingham & Griffith, 2011b

2.5.6 Anchor Failure

A variety of anchorage failures are possible, depending on the details of the connection. For through-bolted anchors, this could include yielding/rupture of the steel components, punching shear failure of the masonry wall around the plate, or failure of the wood joist/blocking to which the anchor is attached. For adhesive-type anchors, there is the additional potential failure mode of pullout/breakout of the anchor and the surrounding masonry. Figure 2.24 provides photos of anchorage failures.



Figure 2.24 – Photo of Anchorage Failures

From: Ingham & Griffith, 2011b

2.5.7 Diaphragm Failure

Recall that URM floor and roof diaphragms are typically constructed of board sheathing (straight or diagonal) which is even more flexible than modern wood diaphragms. Given the low stiffness of the diaphragms, diaphragm “failure” is most commonly associated with excessive deflections, which can lead to collapse of the out-of-plane walls (as the walls lean in/outwards with the deflection). Figure 2.25 shows an aerial view of a building that suffered excessive diaphragm deflections. Observe that the majority of the debris has fallen in the direction of the diaphragm deflection (i.e. into the building on

one side and away from the building on the other). Note that seismic assessment standards typically limit the aspect ratio of diaphragm to 3 or 4 to 1.



Figure 2.25 – Photo of Excessive Diaphragm Deflections

Source: Canterbury Maps (<http://www.canterburymaps.govt.nz/>, retrieved May 2014)

2.6 Retrofitting Measures

Having identified the various manners in which URM buildings tend to fail, we now proceed with identifying manners in which the weaknesses are commonly addressed. Again, this section draws almost exclusively on the work of others (Rutherford & Chekene 1990, 1997, FEMA 2006, Ingham and Griffith 2011b) and the goal is to provide a level of detail suitable for non-technical personnel. The discussion focuses on the various levels of strengthening that are commonly employed. A limited presentation is made on specific details. Refer to the aforementioned documents for further details. For the purposes of this study, three levels of strengthening were identified:

- 1) Fall hazards mitigation (eg. parapet bracing)
- 2) Partial seismic rehabilitation
- 3) Comprehensive seismic rehabilitation

These are general categories that are refined further for engineering purposes. The following sections discuss these three levels of strengthening.

2.6.1 Fall Hazards Mitigation

As shown in section 2.5, parapets, cornices, chimneys, gables, decorative elements, other appurtenances (awnings, mechanical equipment) are potential fall hazards during a seismic event. They tend to be the most vulnerable components and fail at the lowest intensities of shaking. Moreover, they are isolated items and often require only exterior access for strengthening work. As such, they are generally considered the first step in

mitigating URM seismic risk. Indeed, some cities in the United States have had parapet bracing ordinances in place since the middle of the 20th century.

2.6.1.1 Parapets, Chimneys, and Gables

Typical Details

Parapets are the most commonly addressed fall hazard item for URM buildings. The typical solution is to brace the parapets with structural steel and provide roof-wall anchors. Another solution is to replace the parapet with a reinforced concrete cap beam. Figure 2.26 shows a typical parapet bracing detail.

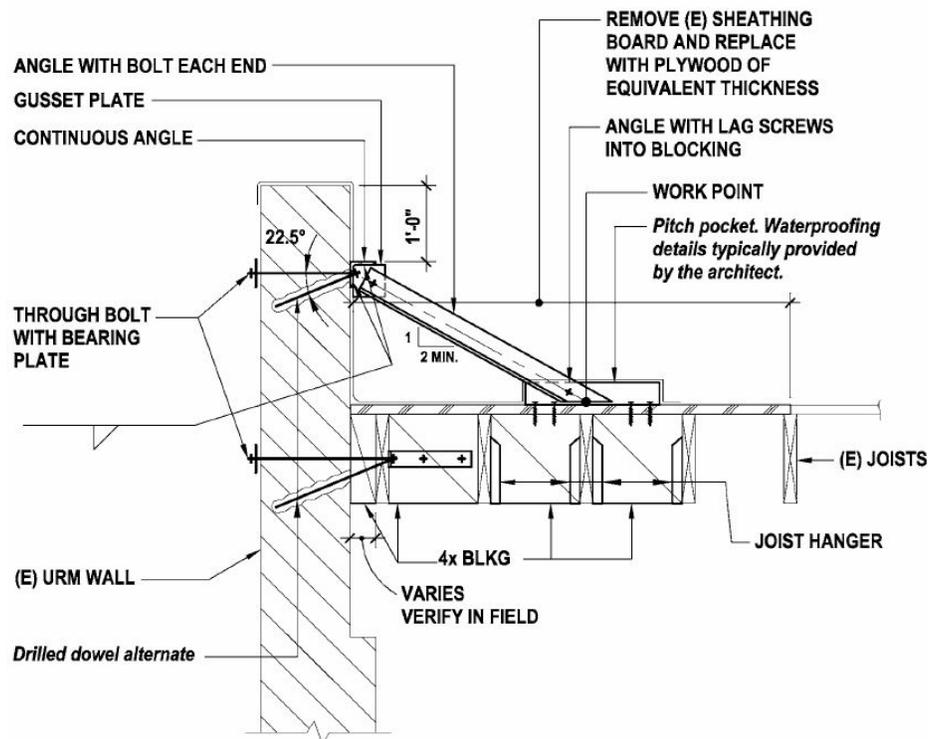


Figure 2.26 – Typical Parapet Bracing Detail

From: FEMA, 2006

The benefits of parapet bracing are twofold:

- The fall hazard is substantially reduced (although not eliminated)
- The wall is now secured at the roof and base, which reduces the likelihood of complete OoP wall collapse, particularly for 1-2 storey buildings

As will be seen in Chapter 4, parapet bracing can significantly reduce the overall damage to the building for low intensity shaking. However, it should be remembered that the remainder of the seismic load path within the building is often incomplete.

Cost & Disruption

One great merit of parapet bracing is its relatively low cost. A typical value³ is about \$300/lin. ft. for thru-bolted connections (see Appendix A for typical costs).

As stated by Rutherford and Chekene (1990), there is little disruption to building occupants. For a typically-sized building (say about 30 feet of parapets along a streetfront), the work can be completed within about a week. Noise-generating work would include drilling/coring of the parapets (for short periods) and operation of power tools for removing and installing lumber. If desired, the work can be completed almost exclusively from the exterior, although this necessitates removal of all roofing and sheathing within 4-6 feet of the parapet. Alternatively, the scope of re-roofing can be minimized at the cost of increased interior access. Figure 2.27 provides photos at various stages of a typical parapet bracing project. Note the bottom left picture in which a “pitch pocket” has been provided to seal the roof penetration. This is no longer typical practice as improved methods have been devised.

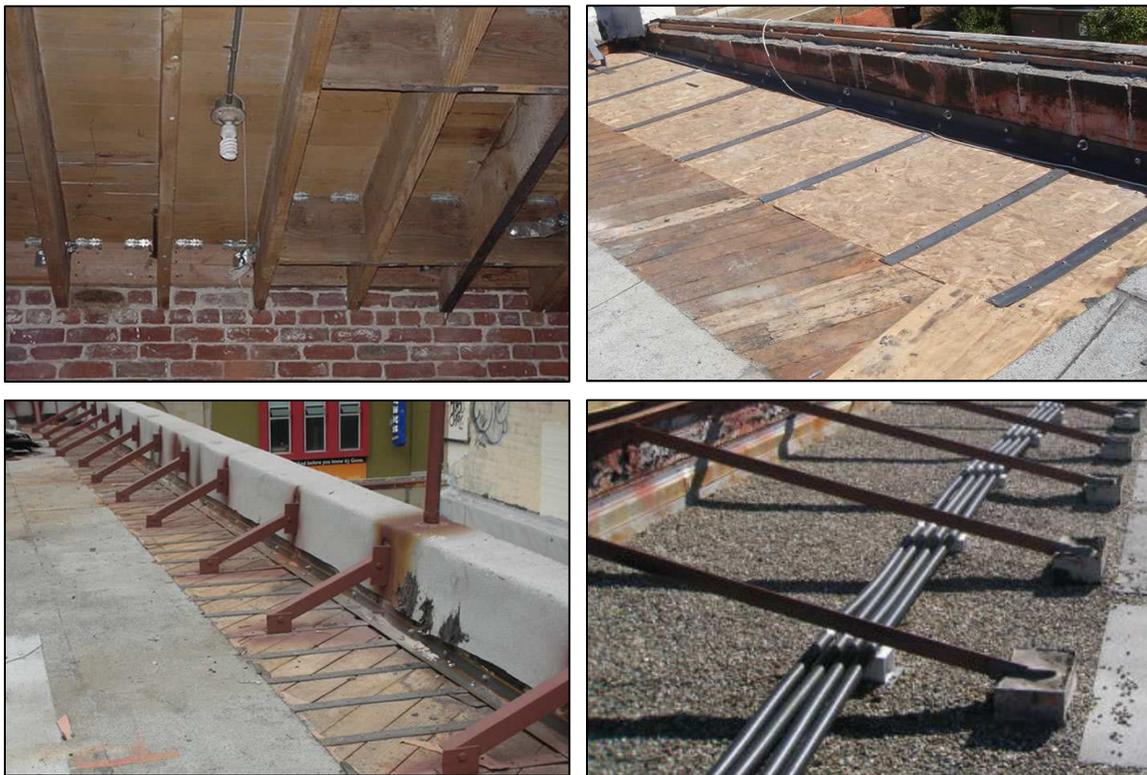


Figure 2.27 – Parapet Bracing Phases – Sheathing/Blocking Installed (Top Left), Roof Anchors Installed (Top Right), Braces Installed (Top Right), Finished Roof (Bottom Right)

Source: <http://www.structuralrenovations.com>

³ Throughout the body of this thesis, costs are presented in 2014 Canadian Dollars (CAD).

Note that if parapets are sufficiently short, the angled bracing may not be required. As per the Canadian (NRC 1992) and American (ICC 2012a) standards, parapets of height-thickness ratios of 1.5:1 or less need not be braced (although roof anchors are still required). This limit is relaxed to 2.5:1 and 4:1 for moderate and low seismic zones, respectively. Chimneys can be treated in a similar fashion, although the work is more localized.

Gable walls are essentially walls of increased height. The top portion of the wall should be braced in a method similar to the above detail for parapets. Anchors are typically also provided at the base of the gable.

Items Sometimes Overlooked

Maintaining the integrity of the roof is an important aspect that is sometimes overlooked. Owners should confirm that the consultants/contractors involved are knowledgeable in this area and that roofing is not simply “left to others” by both parties.

Another key issue sometimes overlooked is the condition of the masonry. In most post-earthquake reconnaissance reports, deteriorated/poor quality mortar is correlated with parapet (and other URM) collapses. There are several methods of assessing the condition of the mortar, from a simple scratch test to (destructive) in-place shear testing. Such testing may represent a slightly or modestly increased cost (say a few hundred to a thousand dollars), but is vital to ensuring the strengthening meets its intended objective. Figure 2.28 provides photos of in-place shear testing, which involves determining the lateral load necessary to initiate shear failure in the bed joints. The process and interpretation of results is commonly discussed in the aforementioned retrofit standards for URM.



Figure 2.28 – In-Place Shear Testing

Photo credit: Read Jones Christoffersen Ltd.

2.6.1.2 Veneers

Typical Details

Veneer courses should be identified and addressed as part of any strengthening (including parapet bracing). Few life safety benefits will be realized if the wall remains standing but sheds its veneer onto the street. Some retrofit standards (ICC 2012a) accept existing corrugated metal ties (as shown in Figure 2.9), provided they meet certain requirements. Of course, the difficulty lies in confirming the presence and condition of these ties, which requires removal of sections of the veneer. This is valuable on larger projects where retrofitting the veneer of an entire building would be extremely costly. For small areas such as a single parapet, it may be more economical to simply provide new ties.

Retrofit ties are typically helical anchors screwed into the backing wythes or steel dowels grouted-in with adhesive. Figure 2.29 shows a typical detail for epoxy veneer anchors. Note that the ties provided in the mortar joints can be hidden by repointing.

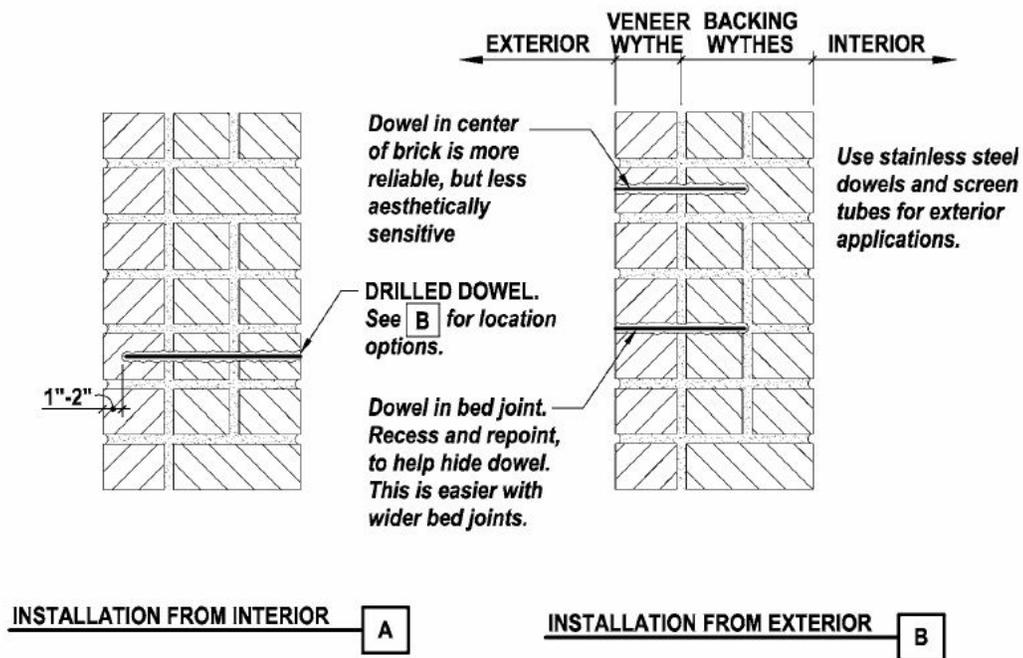


Figure 2.29 – Retrofit Veneer Anchors

From: FEMA, 2006

Cost & Disruption

Veneer anchors are somewhat costly to install, largely due to the cost of access (see Appendix A for costs); a typical unit cost for epoxy dowels is about \$20/sq.ft. of wall surface.

For a small area or a 1-storey building a moveable scaffold platform will suffice. For a larger area, however, the entire building elevation may need to be scaffolded for a period of weeks to months. Interior access can be avoided, but regular drilling of the masonry will produce noise and vibrations that are disruptive to the occupants.

Items Sometimes Overlooked

Veneer anchors are typically quite sensitive to installation workmanship. It is important that the installers are familiar with the requirements specific to the product being used (manufacturers often provide field training for free) and quality control/assurance programs should be implemented.

Some products also specify a minimum quality mortar which should be confirmed by testing. Note that the exterior of a building may have been repointed with a shallow layer of harder, modern mortar; this must not be erroneously taken as being representative of the entire wall.

2.6.1.3 Other Fall Hazards

Typical Details

Other fall hazards include lightweight cornices, and other decorative elements on the building face. The fall hazards are mitigated by providing anchorage into the backing wythes. Figure 2.30 shows a typical detail.

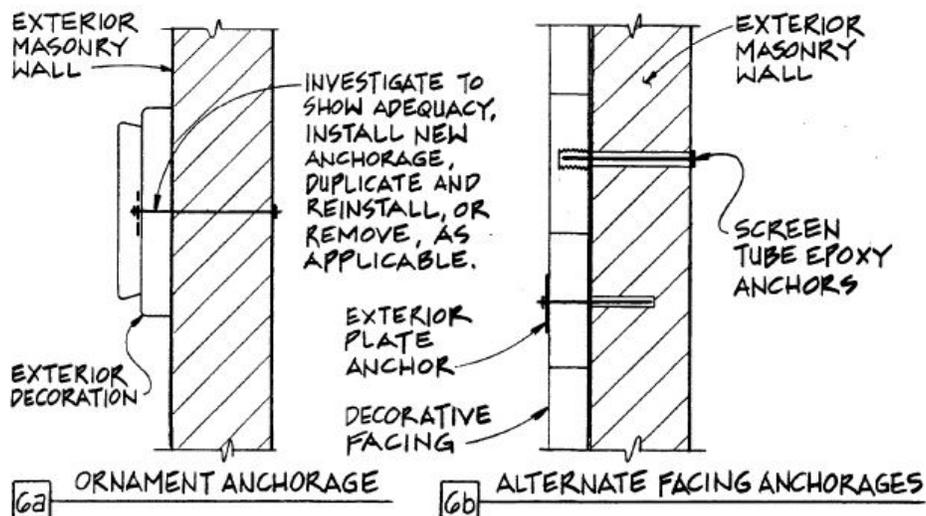


Figure 2.30 – Miscellaneous Fall Hazard Retrofit

From: Rutherford & Chekene, 1990

Cost & Disruption

Cost and disruption are similar to that for veneer anchorage, although the work is somewhat more targeted.

2.6.2 Partial Rehabilitation

Partial rehabilitation is the next step beyond fall hazard mitigation and is specifically geared towards improving the overall behavior of the building and reducing the potential for collapses. In this study, we define partial rehabilitation to include, as a minimum, tension anchors at all floors and roofs (in addition to parapet bracing). It may also include shear anchors, and out-of-plane strengthening for slender URM walls. However, partial rehabilitation measures do not include new elements to increase the in-plane resistance on the walls.

For Canadian practice, the most relevant design standard is the *Guidelines for Seismic Evaluation of Existing Buildings*, as published by the National Research Council of Canada in 1992 (NRC 1992). This standard explicitly recognizes partial rehabilitation by providing a matrix of measures to be included as a function of the seismic hazard, as shown in Figure 2.31.

TABLE A-1 — ELEMENTS OF UNREINFORCED MASONRY TO BE EVALUATED

BUILDING ELEMENTS	Effective Seismic Zone ¹ Z'			
	1	2	3,4	5,6
Masonry		x	x	x
Parapets		x	x	x
Walls anchorage		x	x	x
Walls, h/t ratios			x	x
Walls in-plane shear			x	x
Diaphragms, shear transfer ²			x	x
Flexible diaphragms, demand-capacity Ratios ²				x
Rigid diaphragms ³				x

¹The Effective Seismic Zone, Z', is determined from the NBC Seismic Zone, Z_v, as follows:
 $Z' = Z_v \text{ (NBC)} + 1 \text{ (if } Z_a > Z_v) + 1 \text{ (if } F \geq 1.5)$

²Applies only to buildings analyzed according to the special procedure of Appendix A.

³Applies only to buildings analyzed according to the general procedure of Chapter 3.

Figure 2.31 – NRC URM Seismic Strengthening Matrix

From: NRC 1992

Note that the seismic zones are not compatible with the current edition of the National Building Code of Canada, but effectively areas of low seismic hazard are required to perform little to no strengthening and the required scope increases with increasing seismic hazard. Other examples of partial rehabilitation measures have been incorporated into seismic retrofit ordinances for various cities in the United States, such as San Francisco, Los Angeles, and Seattle; see Chapter 3 for further discussion.

The sections below discuss the individual measures, typical of partial rehabilitation.

2.6.2.1 Tension Anchors

Typical Details

Tension anchors may be either through-bolted or adhesive types and can be installed with access from above or below the floor. The purpose of the tension anchors is to make the OoP walls span between the floors, substantially reducing the potential for collapse. Figure 2.32 shows typical details for tension anchors installed from below and above the floor.

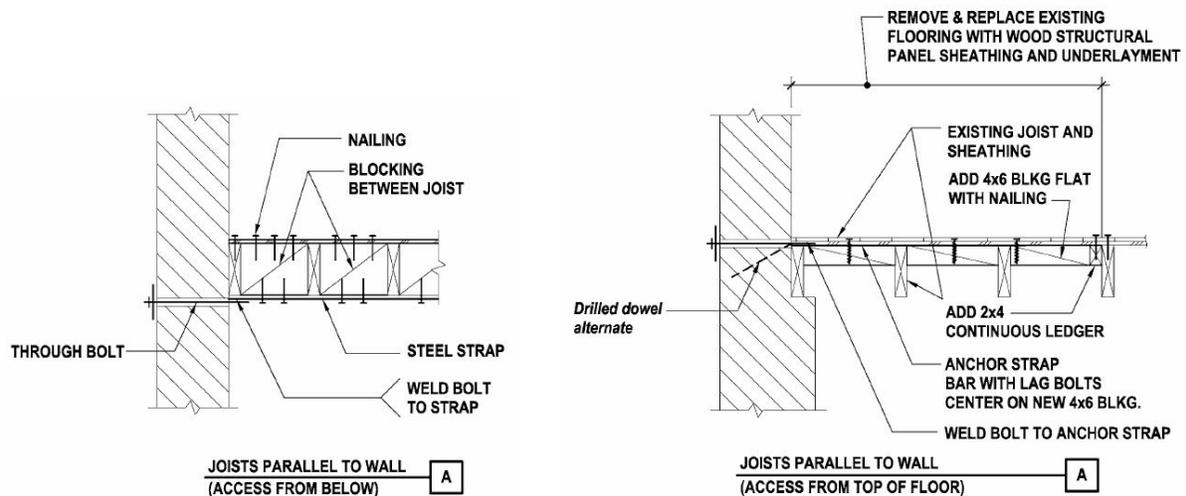


Figure 2.32 – Tension Anchors

From: FEMA, 2006

Cost & Disruption

Tension anchors are a reasonably low cost item, with a typical value of about \$10/sq.ft. of floor area for thru-bolted anchors (see Appendix A). Note that this is intended to be applied to projects in Victoria and is in 2014 Canadian dollars. Of course, this cost is highly sensitive to the finishes, and here it is assumed that expensive finishes such as tile floors need not be removed/replaced. Adhesive anchors tend to be more expensive because the epoxy (or other adhesive) is costly and because the tensile capacity of the anchors is lower (thus more anchors are needed).

The disruption is moderate, as interior access is required. As stated by Rutherford and Chekene (1990), noise and dust would be generated by drilling of the masonry. Building contents within about 4-6 feet of the wall will need to be relocated in many instances. However, this would only be for a short period of time – perhaps a day or two, as the work proceeds from room to room. For residential buildings, this type of work is ideally suited to be completed during tenant turnover.

2.6.2.2 Shear Anchors

Typical Details

Shear anchors must involve some form of grout or adhesive. Grout is a less costly material, but installation is more taxing, because it must be mixed properly and the substrate must be properly prepared (eg. saturated so as not to absorb water from the grout). According to Dizhur et al. (2013), the seismic performance is similar. In practice, grouted anchors are somewhat antiquated and adhesive anchors are the standard. Note that they can serve as tension anchors if properly installed (eg. drilled at an angle, with hold-down and blocking for tension). Figure 2.33 shows a typical shear anchor.

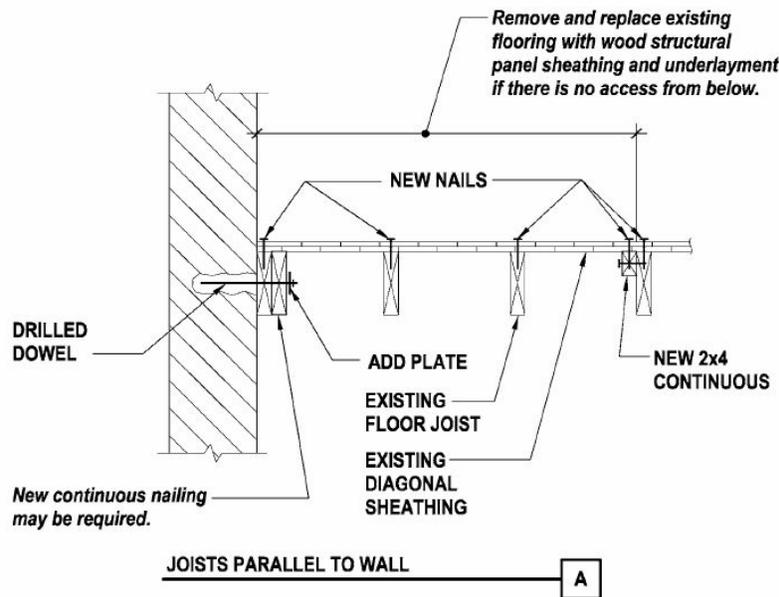


Figure 2.33 – Shear Anchor

From: FEMA, 2006

Cost & Disruption

Cost & disruption considerations are similar to those for tension anchors. An important decision for the owner performing partial rehabilitation is whether to install anchors for just tension, or for both shear and tension. Tension anchors can simply be thru-bolted and so are less costly. However, the combined cost of installing tension anchors and

subsequently also installing shear anchors (as part of a more comprehensive upgrade) would obviously exceed the cost of simply having installed combination anchors to begin with.

2.6.2.3 Out-of-Plane Strengthening

Typical Details

With tension and shear anchors in place, the walls span vertically between the floors under seismic loading. However, they can crack, rock, and subsequently collapse if the shaking is of sufficient intensity and duration. Out-of-plane strengthening addresses this failure mode. The most common solution is structural steel “strongbacks” installed at regular spacing along the wall, which act as a splint to help prevent the out-of-plane cracking and rocking. Another common solution is an “intermediate brace” which helps to reduce the unsupported height of the wall. Figure 2.34 provides a detail for each of these solutions and Figure 2.35 provides photos. Other solutions could include shotcrete overlays and reinforced/post-tensioned centercores (see FEMA 2006).

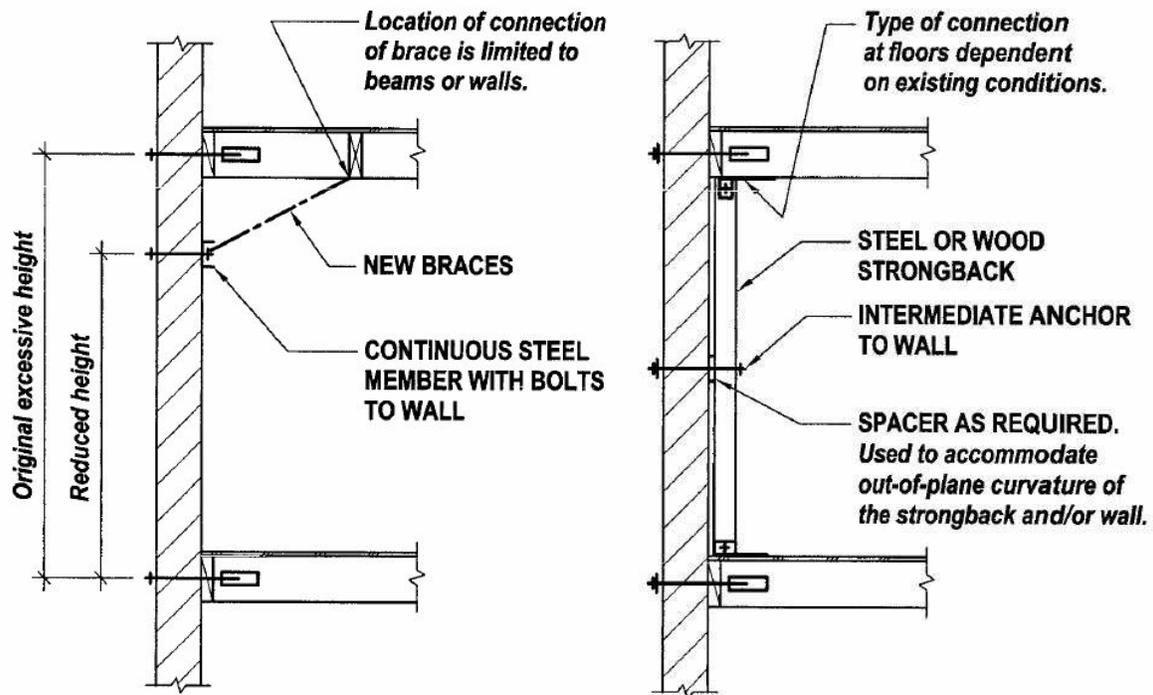


Figure 2.34 – OoP Bracing, Intermediate Brace (left) and Strongback (right)

From: FEMA, 2006



Figure 2.35 – Photos of Intermediate Braces (Left) and Strongbacks (Right)

Photo credit: Read Jones Christoffersen Ltd.

Cost & Disruption

Providing out-of-plane strengthening (in addition to parapet bracing, and shear and tension anchors) represents a significant increase in cost. A typical unit cost (in Victoria in 2014 dollars) is about \$18-25/.ft. (see Appendix A). Disruption is also significantly increased: as large structural steel components are involved, the contractor will require somewhere to store them on site. Removal of wall finishes may also be necessary.

2.6.3 Comprehensive Rehabilitation

Comprehensive rehabilitation includes all the measures from partial rehabilitation (which effectively address OoP collapse of URM walls), but also addresses in-plane demands on walls and generally ensures a complete seismic load path. This often involves introducing new seismic resisting systems, such as structural steel frames or reinforced concrete shear walls.

Several design standards/guidelines can be applied. These include the following:

- Seismic Evaluation Guidelines for Existing Buildings (NRC 1992)
- The International Existing Buildings Code (ICC 2012a)
- ASCE 41-13: Seismic Evaluation and Retrofit of Existing Buildings (ASCE 2013)

Where existing URM walls are insufficient, traditional design practice is to provide a new seismic force resisting system - designed to some fraction of current code forces - and to ignore the resistance and deformation compatibility of the in-plane URM walls. In Victoria, 70% of current code is commonly specified by the local building authorities for seismic upgrading of URM buildings.

In the United States, "performance-based" standards such as ASCE 41 are becoming increasingly common in seismic evaluation and upgrading of buildings, as the building authorities begin to specify these standards. Rather than specifying a "percent code" force level, these standards set performance criteria for individual components (measured in displacements, accelerations, or some other indicator of damage/safety) and then the retrofit is designed to ensure these criteria are met.

Such standards have several scientific merits over the traditional, prescriptive approaches (eg. percent of code). Perhaps the most important merit is that they explicitly provide several performance levels that could be targeted and thus require a dialogue with the owner and other stakeholders as to the expected seismic performance of the building. Figure 2.36 below shows a spectrum of possible performance levels from ASCE 41. As can be seen, performance goals could range from "very little damage" to "barely standing."

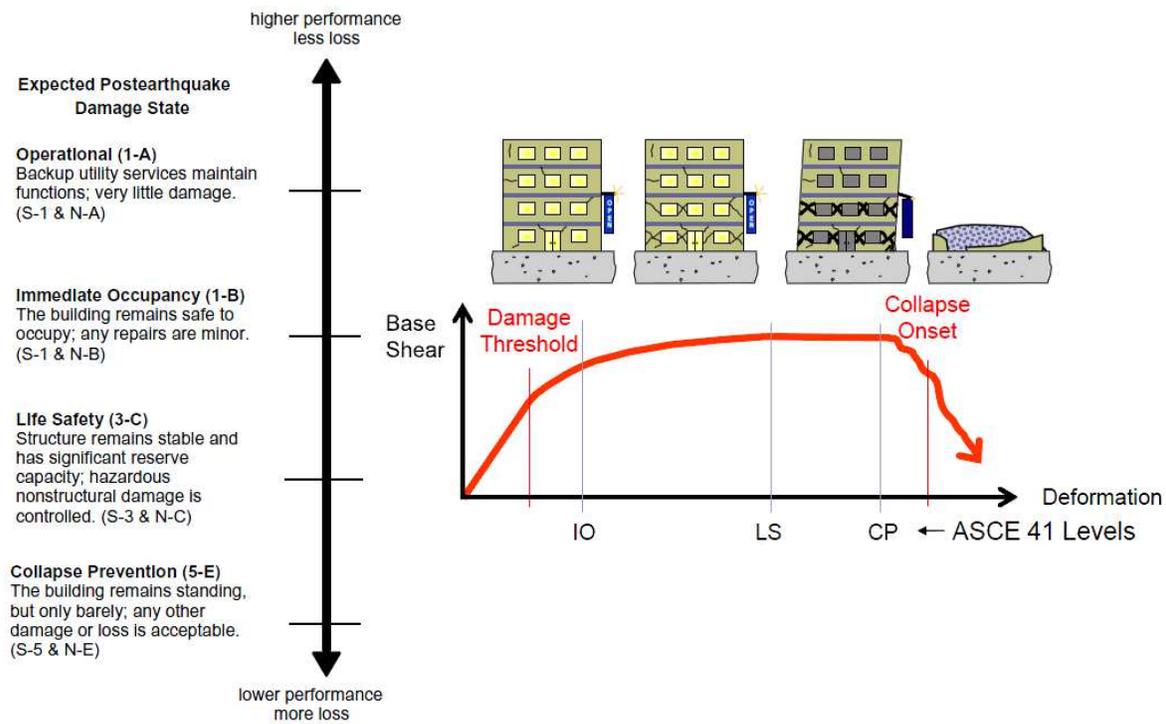


Figure 2.36 – Seismic Performance Levels

Modified From: ASCE, 2013

Note that setting a performance goal also involves specifying under what earthquake intensity a given level of performance should be achieved (eg. under a code-level earthquake).

Traditional design practice for retrofitting could be considered as targeting the “life safety” performance objective at the specified design force demands (eg. 70% of current code), since this is the objective of codes for new buildings. In the engineering community, it is well-known that such designs make no attempt to prevent a building from being damaged beyond economical repair as a result of a design-level earthquake.

As noted by Rutherford and Chekene (1997), achieving higher performance levels such as Immediate Occupancy is very difficult for URM buildings and would likely necessitate nearly reconstructing the entire structure. Nonetheless, it is important to explicitly specify the expected performance, as many building owners have spent hundreds of thousands or even millions of dollars on earthquake strengthening, only to have the building demolished after an earthquake - not because of safety concerns but because repairing the damage was not economically feasible.

In general, a comprehensive rehabilitation will necessitate providing a largely new seismic force resisting system at some locations. Regardless of the desired level of performance, many of the new elements introduced will be similar in form (although the size, location, number, and detailing may differ).

2.6.3.1 New Seismic Force Resisting Elements

Typical Details

New seismic force resisting elements are employed at open fronts of row buildings (where URM walls are essentially absent) and where existing URM walls have insufficient resistance. For particularly elongated buildings, new elements may also be required in the interior (rather than just along exterior walls). Where existing URM walls are insufficient, traditional practice is to design the new elements to some fraction of current code forces and ignore the resistance (and deformation compatibility) of the URM walls.

In Victoria, steel concentrically braced frames are by far the most common design solution and so we focus on those herein. Many other elements such as reinforced concrete/masonry shear walls or steel/concrete moment frames are possible. Rutherford and Chekene (1990) provides a summary of all the common elements. Figure 2.37 shows an example illustration of steel braced frames in a URM retrofit applications and Figure 2.38 provides an example photo.

Cost & Disruption

The cost to complete this type of retrofit (in Victoria, in 2014 dollars) is about \$40/sq.ft. (see Appendix A), which includes all elements of a comprehensive retrofit. This is just the cost of the seismic retrofit work. Because of the cost and disruption, architectural renovations are often combined with the strengthening.

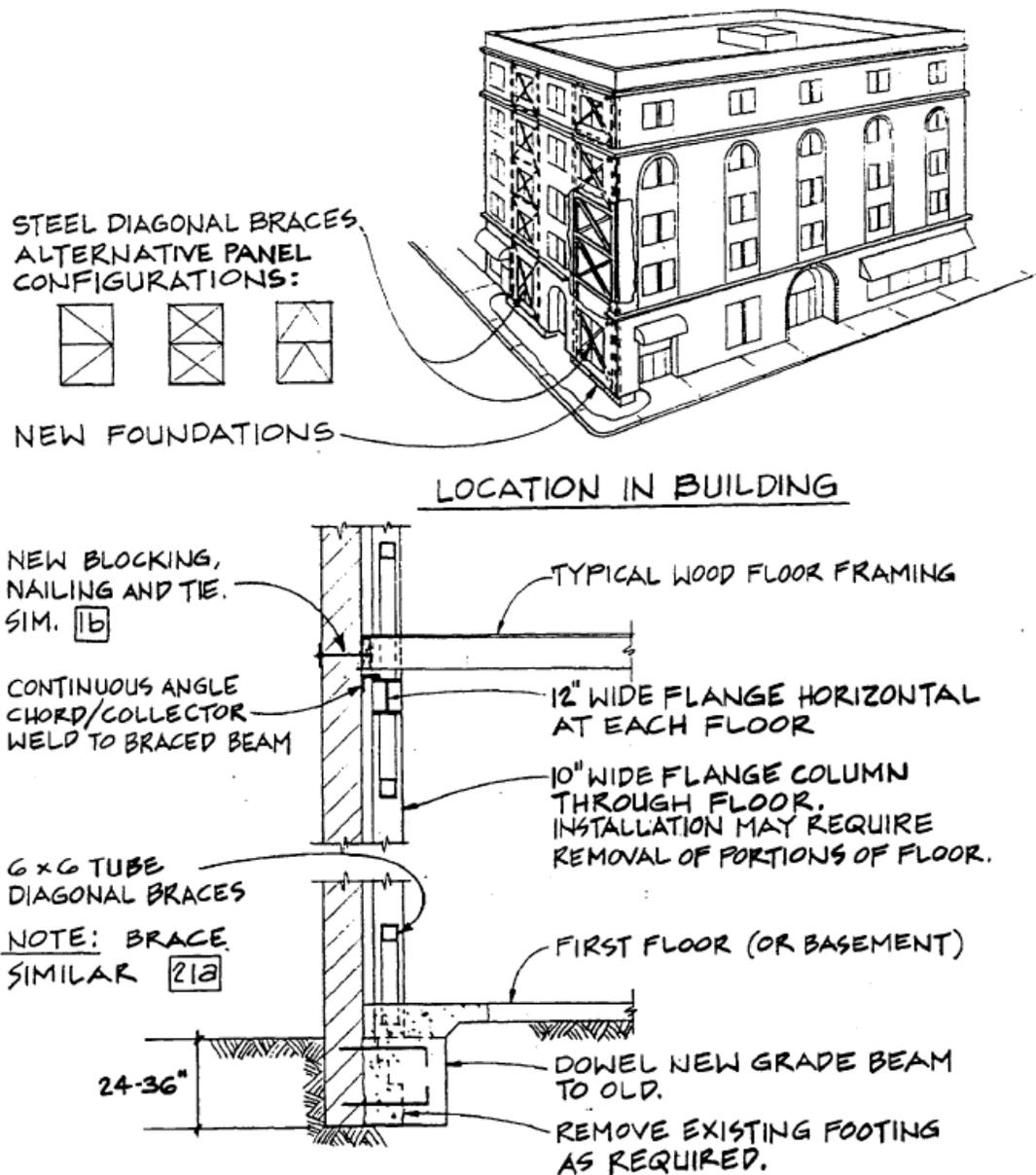


Figure 2.37 – Illustration of URM Retrofitted with Steel CBF

From: Rutherford and Chekene, 1990



Figure 2.38 – Example Photo of URM Retrofitted with Steel CBF

Photo credit: Read Jones Christoffersen Ltd.

The work involved with this level of strengthening can be quite disruptive. As stated by Rutherford and Chekene (1990), workers are often on site for several months and large areas would need to be turned over to the contractor for several weeks at a time. The logistics of actually getting the steel into the buildings and erected can be challenging and often the most effective way to do so is to cut an access hole in the roof and make use of a mobile crane. Holes in floors also must be cut to accommodate the columns.

In general, steel moment frames involve a similar level of disruption. Of course, reinforced concrete or masonry shear walls eliminate the need for cranes and unwieldy steel members, but the work generates much more dust. Wood shear walls are likely the most appealing from a disruption and constructability point of view, but their capacity is limited and thus they are most commonly employed in the interiors of buildings to reduce excessive diaphragm spans.

2.6.3.2 Diaphragm Strengthening

Typical Details

The flexible wood diaphragms common to URM buildings have a profound impact on their dynamic response. Depending on the diaphragm stiffness, the accelerations at a floor can be increased or decreased and the out-of-plane rocking response of URM walls is also heavily affected by diaphragm flexibility (Penner 2013). Depending on the properties of the diaphragms and out-of-plane walls to which they are connected, diaphragms may be found to have insufficient strength or stiffness.

The typical method of strengthening diaphragms is by installing plywood, either atop the existing floor sheathing or to the underside of the joists. Steel sheet metal straps can also be used to strengthen existing wood diaphragms, or the diaphragm could effectively

be replaced by installing new structural steel horizontal bracing. Finally, a concrete overlay can be applied. Wilson (2012) discusses in detail the seismic behavior and the strengthening of wood diaphragms in URM buildings. Figures 2.39 and 2.40 show design details and provide an example photo, respectively, for board sheathing retrofitted with a plywood overlay. An unblocked retrofit solution is shown, but blocking can be provided in cases where additional strength and stiffness are required.

It should be noted that certain Canadian and American design standards (NRC 1992, ICC 2012a) are limited to flexible diaphragm buildings, and so design solutions that render the diaphragm rigid are often not preferable.

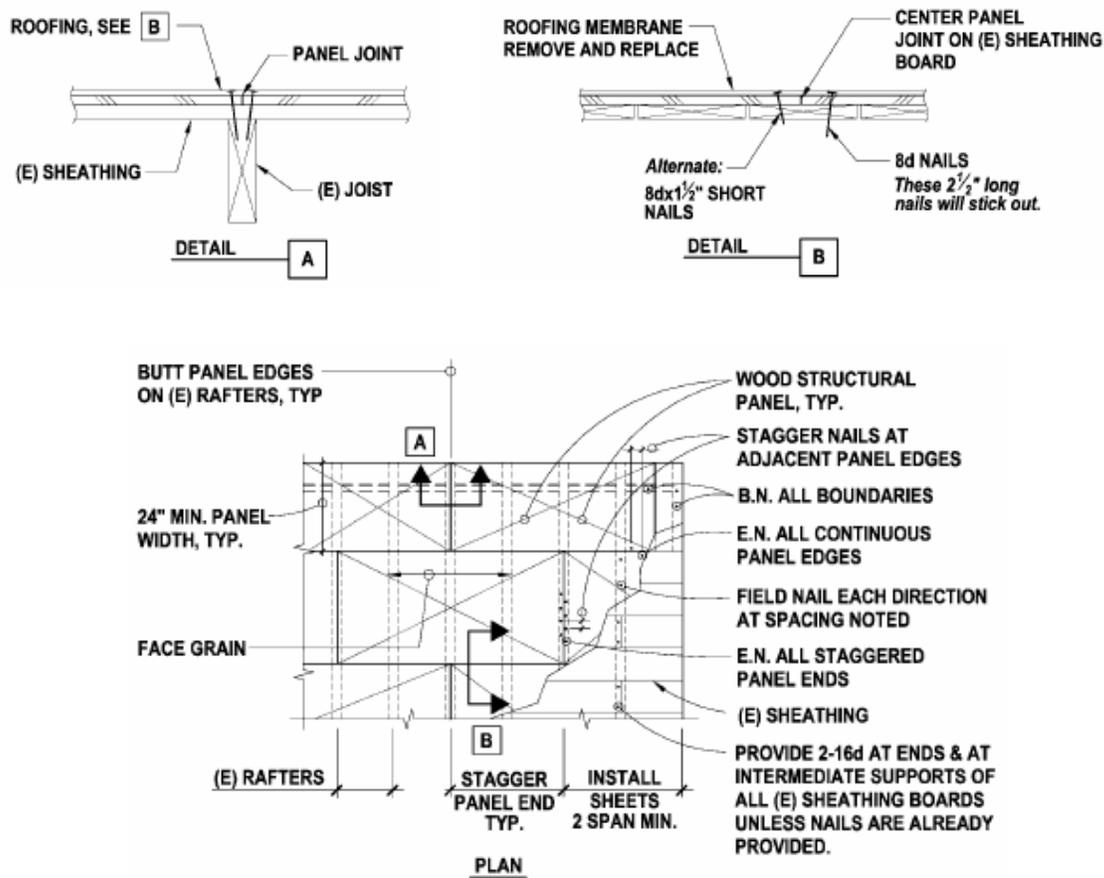


Figure 2.39 – Illustration of Plywood Overlay Retrofit

From: FEMA, 2006



Figure 2.40 – Diaphragm Strengthening

From: Wilson, 2012

Cost & Disruption

Diaphragm strengthening is typically performed only as part of a comprehensive rehabilitation. Much of the cost associated with diaphragm strengthening is due to removal and reinstatement of finishes and other building contents. As such, a cost for diaphragm strengthening alone is not provided.

The level of disruption associated with diaphragm strengthening is high, since floor or ceiling finishes must be removed and building contents must be relocated. As such, it is common practice to design retrofits so as to avoid the need for diaphragm strengthening. Similarly, it is noted that many URM seismic evaluation/strengthening methodologies, including the Canadian guideline (NRC 1992), only require diaphragm evaluation for buildings in the highest seismic hazard zones.

2.7 Summary and Conclusions

In this chapter, an introduction to unreinforced masonry (URM) buildings was provided. The form and components of a “typical” building on the west coast of North America were reviewed, and buildings in Victoria, BC were found to be similar. The structural idealization and dynamic behavior of such buildings was presented and it was noted that their relatively rigid walls and flexible diaphragms result in seismic behavior that is very different from that assumed for most modern buildings in new building codes. Finally, common seismic deficiencies and typical corresponding retrofit solutions were presented.

Chapter 3

URM Seismic Risk Mitigation Programs

3.1 Purpose and Scope

Despite their well-known seismic vulnerability, URM buildings continue to be a leading (structural) source of loss of life and property damage in earthquakes and mitigation programs are routinely met with substantial public and private resistance, primarily due to the cost of seismic upgrading. The purpose of this chapter is to review the historical impetus for URM seismic risk mitigation, and the programs that have been implemented. The efforts to date in Victoria, BC will be compared to those abroad.

3.2 The Impetus for URM Seismic Risk Mitigation

Historically, observations of poor seismic performance have been the primary driving force behind URM seismic risk mitigation. Two interestingly similar examples include the 1931 Hawke's Bay earthquake in New Zealand and the 1933 Long Beach earthquake in California, USA: unreinforced masonry construction was effectively banned in both of these locations, in direct response to the resulting losses.

The following statistics provide a sampling of the seismic performance of URM buildings in various significant historical earthquakes. Examples of both poor performance of unstrengthened buildings and good performance of strengthened buildings are provided.

- **1925 Santa Barbara:** 40% of unstrengthened buildings suffered severe damage or collapsed (FEMA 2009)
- **1933 Long Beach:** 20% of unstrengthened buildings suffered severe damage or collapsed (FEMA 2009)
- **1971 San Fernando:** 49 deaths caused by collapse of URM buildings at Veteran's Administration Hospital (Stover and Coffman 1993)
- **1983 Coalinga:** 60% of unstrengthened buildings suffered severe damage or collapsed (FEMA 2009)
- **1989 Loma Prieta:** in regions of Modified Mercalli Intensity VIII (generally within 50km of the epicenter), 40% of unstrengthened buildings were demolished; additionally, 9 deaths were attributed to URM (Lizundia, Dong and Holmes 1993)

- **1994 Northridge:** of about 7000 URM buildings in the Los Angeles area (which experienced mostly MMI VII shaking), only about 600 were unstrengthened at the time of the earthquake; no fatalities due to URM buildings were recorded (Bruneau 1995)
- **2009 L'Aquila:** 39 deaths attributed to URM, representing approximately 20% of the total deaths toll. The remainder were due to high population density in a few reinforced concrete buildings that collapsed (Alexander and Magni 2013)
- **2011 Christchurch:** 39 deaths attributed to URM (Canterbury Earthquakes Royal Commission 2012); 82% of URM buildings red-tagged and only 1% green tagged (Ingham and Griffith 2011b); no public access permitted to many areas of central business district for over a year

While tragic, these losses have played a vital role in the advancement of URM seismic risk mitigation policies. The following sections review the seismic risk mitigation efforts to date by various jurisdictions (eg. city, state, province, or country) facing similar seismic risks as Victoria, BC. Seismic retrofit ordinances and incentive programs are discussed.

3.3 United States

The United States has been at the forefront of seismic risk mitigation, especially in California and (more recently) throughout the Pacific Northwest. This section summarizes seismic upgrading policies and financial aid/incentive programs for selected jurisdictions. Note that this is not an exhaustive presentation and additional information is available in the literature.

3.3.1 California

This section reviews the seismic risk mitigation efforts in the cities of Los Angeles, San Francisco, and Palo Alto. The focus is on strengthening of the general existing stock, through risk mitigation ordinances. Measures such as the 1933 Field Act, 1939 Garrison Act, and the 1967/1968 Greene Acts were significant steps for the seismic risk mitigation of new and existing school buildings (many of which were of URM construction). Rutherford and Chekene (1997) provides a succinct summary.

3.3.1.1 Los Angeles

The City of Los Angeles was the first local government to pass a retroactive URM seismic ordinance, in the form of its 1949 parapet correction ordinance (Rutherford & Chekene 1997). In 1981, the City adopted an ordinance for comprehensive seismic

strengthening, which is now known as Division 88 (City of Los Angeles 1985). It covered all URM bearing wall buildings (i.e. infill buildings were not included), except one and two family dwellings and apartments with four or less units (Rutherford & Chekene 1997), which totalled over 8000 buildings.

Varying timelines for compliance were established based on a “rating classification.” The rating classification is based on building function and occupant load, as shown in Table 3.1.

As expected, higher priority buildings had shorter timelines for compliance: essential buildings were to be strengthened within three years of their owners being served notice from the City that their building fell within the scope of the ordinance. Lower priority buildings had the option of extending the deadline (as shown in Table 3.1) for full compliance by performing partial retrofits. Partial retrofits included parapet bracing and tension anchors.

Table 3.1 – Division 88 Compliance Timelines

Rating Classification	Definition	Full Compliance (no partial retrofit)	Full Compliance (with partial retrofit)
Essential Building (Highest Priority)	Medical/Emergency Services Centers	3 years	3 years
High-Risk Building	>100 Occupants	3 years	3.25 years
Medium-Risk Building	All Others	3 years	4-6 years (dep. on occupant load)
Low-Risk Building (Lowest Priority)	<20 Occupants	3 years	7 years

The City of Los Angeles did not provide any significant incentives for the general building stock. However, the City’s Community Development Department (CDD) provided low-interest loans to cover project costs for residential and mixed-use buildings. Statistics on the total number of buildings covered were not available, but there were over 1500 residential or mixed-use URM buildings affected by Division 88. Additionally, the City’s Rent Stabilization Division controlled rent increases (Comerio 1989). It should be noted that in order to be eligible for the financing, buildings also had to receive basic fire safety upgrades such as sprinklers and egress equipment.

While low-interest financing is a useful option, Comerio (1989) notes a few pitfalls that were encountered:

- The funds were intended only for seismic upgrading and the minimally required architectural and fire safety work, but several building owners took advantage of the loans to complete other work; thus, a strict control system was needed.
- Changes in the work during construction were reviewed by the Building and Safety Department, which tended to slow construction; it was recommended that measures be put in place to expedite this process.
- By 1989, 8% of the 1500+ residential/mixed-use buildings had been demolished and another 9% was in danger of demolition due to non-compliance with the ordinance; it was recommended that the City implement some type of demolition control, including requirements that an owner at least obtain and submit cost estimates for strengthening and that the owner meet with the City to discuss funding options.
- Many of the buildings housed low-income tenants, who already spent an above average portion of their income on rent; it was recommended that rent increases be limited to \$100/month or less (existing rental rates were \$400-500/month).

While ultimately quite effective, the ordinance was fiercely contested and was debated in political arenas for over eight years (1973-1981). Detractors argued (rightfully so to at least some extent) that the ordinance would place pressure on poor, marginalized citizens of the city through displacement and increased rent (Alesch and Petak 1986). With regard to the compliance rate of the program, the California Seismic Safety Commission (CSSC) provided the following figures in 2006.

Table 3.2 – Los Angeles URM Ordinance Compliance Rates

Total URM s	Historic URM s	% Strengthened	% Demolished	% Non-Compliant
9211	255	67%	21%	12%

With regard to the seismic performance of the retrofits, the 1994 Northridge earthquake provided a significant test. Shaking in Los Angeles was mostly of MMI VII (PGA=0.15-0.35g). At this time, only about 600 of 8200 URM buildings had yet to be mitigated (Bruneau 1995). Bruneau noted that for unstrengthened buildings, out-of-plane failures were numerous. However, no lives were lost since the earthquake occurred at 4:30am, when the streets were largely empty. The majority of strengthened buildings survived undamaged, although about 200 of 6000 suffered moderate to severe damage. As noted by Bruneau (1995), this performance actually exceeded prior expectations (judgmentally) established by a panel of experts (EERI 1994).

3.3.1.2 San Francisco

In 1976, the City/County of San Francisco enacted its Parapet Safety Program, which required owners to retain a structural engineer to provide a seismic assessment of the parapets of their building. The ordinance applied to all pre-1949 URM buildings that posed fall hazards to public sidewalks or occupied spaces (Bonnevill and Cocke 1991) and bracing requirements were as per the engineer’s assessment, to a prescribed force level.

In 1986, Senate Bill 547 (commonly known as “the URM law”) was passed, which required jurisdictions in Seismic Zone 4, the highest seismic zone, to (CSSC 2006):

- 1) Identify all existing unreinforced masonry buildings by the end of 1989
- 2) Develop a risk mitigation program
- 3) Report on progress to the California Seismic Safety Commission

The law allowed each jurisdiction to develop its own mitigation program, which could range from simply informing owners that their buildings appeared to be of URM construction to mandatory comprehensive seismic upgrading of all subject buildings. See 3.3.1.4 for further discussion on the URM law.

Like many communities, San Francisco opted to employ a mandatory strengthening program. In 1992, it passed ordinance 225-92, which mandated strengthening/abatement of approximately 2000 identified URM buildings. Similar to the Los Angeles ordinance, various timelines for compliance were established based on levels of risk. Table 3.3 shows the selected compliance deadlines.

Table 3.3 – Ordinance 225-92 Compliance Timelines

Risk Level	Definition	Apply for Building Permit	Construction Complete
Level 1	Group A occ. (300+ occupants) Group E occupancies 4+ storey buildings on poor soil	2.0 years	3.5 years
Level 2	Non-Level 1 buildings located poor soils (in certain high-density areas, such as downtown)	2.5 years	5.0 years
Level 3	Non-Level 1 buildings located on poor soil in other areas (i.e. lower density)	8.0 years	11.0 years
Level 4	All other URM buildings	10.0 years	13.0 years

Strengthening for most buildings was essentially to be in conformance with the 1991 Uniform Code for Building Conservation (ICBO 1991), which was largely based on an extensive testing and research program completed in the 1980’s by three prominent engineering firms in California (ABK 1984). Extensive discussion is provided elsewhere on the technical basis (Rutherford & Chekene 1990, 1997, Bruneau 1994), but here it is simply mentioned that the methodology was created to reduce the required structural interventions and thus is thought to provide a reduced level of safety as compared to new buildings codes of the day.

Ordinance 225-92 also contained a relaxation for certain residential and commercial buildings: eligible buildings were able to meet the requirements of the ordinance by performing a “bolts-plus” retrofit, which called for just diaphragm-to-wall connections (shear and tension) and out-of-plane bracing for walls exceeding height-to-thickness limits, as specified in the UCBC (SFBIC 2013). Table 3.4 shows the various measures included in the various strengthening schemes discussed thus far.

Table 3.4 – Measures Included in Various Strengthening Schemes

Modified After: Bonneville and Cocke, 1991

Strengthening Measure	Parapet Ordinance	Bolts-Plus	UCBC	Code for New Buildings
Parapet Bracing	X ¹	X	X	X
Roof-to-Wall Tension Anchors	X ¹	X	X	X
Diaphragm-to-Wall Anchors (Shear and Tension)		X	X	X
Out-of-Plane Wall Bracing		X	X	X ²
In-plane Wall Strengthening			X	X ²
Diaphragm Strengthening			X	X ²
Veneer Ties		X ³	X ³	X ³
Other Rooftop Fall Hazards (eg. cornices, chimneys)			X ³	X ³

¹ Required only at locations where parapets pose a public fall hazard (eg. walkways, entrances, locations overlooking buildings below)

² More stringent requirements than those of the Uniform Code for Building Conservation (UCBC)

³ Not historically well-enforced in some jurisdictions

The full provisions of the UCBC are considered to provide a lower level of life-safety than new building codes and the “bolts-plus” provisions must obviously achieve less. Nonetheless, the pressing socioeconomic issues associated with mandatory strengthening led to the bolts-plus provisions (FEMA 1998).

It should be noted that, in order to qualify for the bolts-plus procedure, buildings were required to meet a number of criteria, including:

- The building does not contain occupancies of group A (assembly) with > 300 persons, group E (education), group H (hazardous), or group I (industrial)
- Mortar shear strength (from in place shear tests) ≥ 30 psi
- Wood diaphragms at all levels above the base of the building
- Maximum of 6 storeys
- The building does not have various irregularities, listed below
 - Soft/weak storey
 - In-plane discontinuity (of walls)
 - Diaphragm discontinuity
 - Out-of-plane offsets
- Minimum of two lines of lateral force resisting elements in each direction (i.e. open front buildings do not qualify); solid wall must comprise at least 40% of the walls length to be considered a line of resistance
- The building has or will be provided with crosswalls at a spacing not exceeding 40 feet on center

Owners could either correct deficiencies and rehabilitate to bolts-plus or implement a UCBC-compliant retrofit scheme. These requirements are relatively restrictive: in a review of 66 red-tagged San Francisco buildings after the Loma Prieta earthquake, Bonneville and Cocke (1991) estimated that only 35 would have qualified for a bolts-plus retrofit.

In terms of incentives, low-interest loans were made available through a \$350 million general obligation bond, approved by a public vote. \$150 million of this was devoted to affordable housing, while the remainder was available for any building (City of San Francisco n.d.).

With regard to the compliance rate of the program, the California Seismic Safety Commission (CSSC) provided the following figures in 2006.

Table 3.5 – San Francisco URM Ordinance Compliance Rates

Total URM	Historic URM	% Strengthened	% Demolished	% Non-Compliant
1976	516	78%	8%	14%

In comparing these figures to those of Los Angeles, we can see that San Francisco achieved a similar compliance rate, but appears to have had more success in preventing demolitions. This is likely due to a number of factors, including the following:

- The ordinance allowed a relaxation for certain buildings
- Design standards and construction methodologies were more refined, resulting in lower costs to upgrade
- Loans were made available to all buildings, not just those of residential/mixed-use occupancies
- A much higher fraction of San Francisco’s buildings (26%) were considered historic than those of Los Angeles (3%) (CSSC 2006)
- San Francisco was likely able to take advantage of lessons learned in Los Angeles in terms of demolition control

Although mandatory strengthening measures were not yet in place, the 1989 Loma Prieta earthquake provided a test of the parapet strengthening measures. Based on damage observed in their post-earthquake safety evaluations, Bonneville and Cocke (1991, 1992) assessed the effectiveness of the parapet bracing. Their observations were as follows:

- 66 red-tagged URM buildings were included in the study
- 50 of these had parapet bracing – none experienced collapse of parapets or out-of-plane walls
- 16 of these did not have parapet bracing – 3 suffered partial collapse, one of which caused 5 deaths (Wiggins, Breall and Reitherman 1994)

Based on the extents and types of failures observed, Bonneville and Cocke also projected the performance that bolts-plus retrofits would have provided and concluded that “no collapse would have occurred if all URBs had undergone the equivalent of a bolts-plus strengthening.” Of course, it was also noted that the good performance of these limited strengthening measures was largely because shaking was of only moderate intensity.

3.3.1.3 Palo Alto

The City of Palo Alto implemented a different, but reasonably successful, URM seismic risk mitigation program. After a failed attempt to establish a mandatory program in 1982 – and the Coalinga earthquake of 1983 – the City created a “Seismic Hazard Committee.” The committee represented a variety of stakeholders and was tasked with developing an acceptable risk mitigation program. In 1986, the City passed an ordinance that entailed the following (FEMA 1998, Gibson Economics 2014):

- Three buildings types were included
 - Unreinforced masonry
 - Pre-1935 non-URM with 100+ occupants
 - Pre-1976 non-URM with 300+ occupants
- Building owners were required to engage a structural engineer to conduct a seismic evaluation of the building, specifying the necessary upgrades
- Seismic evaluations were submitted to the City and owners were required to inform building occupants that the reports were available for their review
- Within one year of filing the report, owners were required to submit a letter to the City indicating their intentions to address the building

Strengthening remained voluntary and incentives were made available, including FAR (floor/lot area ratio) increases of up to 25%. However, the incentives were not widely used (Gibson Economics 2014). The primary driving factor was the public and occupant awareness created by the publicly available engineering assessments. For example, the level of awareness was so high that seismic improvements were able to be marketed by building owners. Additionally, some tenants agreed to help finance upgrade costs and other voluntarily agreed to vacate the space during construction and return upon completion (FEMA 1998). Table 3.6 provides the compliance statistics as of 2006.

Table 3.6 – Palo Alto URM Ordinance Compliance Rates

Total URM	Historic URM	% Strengthened	% Demolished	% Non-Compliant
47	4	43%	21%	36%

The compliance rate (strengthened + demolished) is much higher than other voluntary programs which average 24% (CSSC 2006). Aside from the program itself, however, there were a number of factors which likely played a role, the three most significant being the following:

- The relatively small size of the community (1990 population of 55,000), which facilitated community involvement and generation of support for the program
- The relatively small size of the URM building stock, leading to smaller overall costs
- The fact that the community was rather affluent, being home to Stanford University, many political figures, and wealthy high-technology professionals (FEMA 1998)

Nonetheless, this program is an example of policy-making that was carefully crafted to generate public support rather than opposition, making it successful from both an engineering and a political view.

3.3.1.4 Effectiveness of Mandatory versus Voluntary Strengthening

Having reviewed a few successful programs in detail, we now take a step back and examine the broader question of the relative effectiveness of mandatory versus voluntary strengthening for URM buildings, as it was experienced in the State of California.

In 1986, the State of California passed Senate Bill 547, more commonly known as the “URM Law” (California Legislature 1986). The law applied to approximately 26,000 URM buildings in areas of highest seismic hazard under the California Building Code (based on the 1985 Uniform Building Code at the time the law was passed) and required 365 affected local governments to: inventory URM buildings, establish loss reduction programs, and report on progress (CSSC 2006). The law recommended, but did not require, that local governments include mandatory strengthening in their loss reduction programs. “Voluntary strengthening” and “notification-only” programs also met the requirements of the law.

Mandatory strengthening programs generally required comprehensive upgrading for in-plane and out-of-plane seismic demands. The most commonly used standard was Appendix Chapter 1 of the Uniform Code for Building Conservation (ICBO 1991). Other mandatory programs, such as San Francisco’s, required only partial retrofits for some buildings to limit economic impacts on the community.

Voluntary strengthening programs typically encouraged comprehensive seismic upgrading, similar to the mandatory requirements noted above.

Notification-only programs typically included only a letter from the local authority having jurisdiction to buildings owners, stating that their building is of URM construction and is a potential seismic risk.

Of the 365 affected local governments, 283 were found to have URM buildings in their jurisdiction. The majority of these adopted mandatory strengthening programs. Table 3.7 summarizes the loss reduction program types established and affected number of URM buildings as of 2006 (CSSC 2006):

Table 3.7 – California URM Loss Reduction Program Statistics

After: Paxton, Elwood, Barber & Umland, 2013

Program Type	# Jurisdictions	% Jurisdictions	# URMs	% URMs
Mandatory Strengthening	134	47%	19,043	73%
Voluntary Strengthening	39	14%	1,269	5%
Notification-Only	46	16%	1,487	6%
Other ¹	41	15%	3,737	14%
No Program Established	23	8%	409	2%
Total	283	100%	25,945	100%

¹ Combinations and variants of the above noted programs

As of 2006, approximately 55% of the affected URM buildings had been retrofitted and 15% had been demolished, for an overall mitigation rate of 70%. Table 3.8 shows a breakdown of the results for each program type. The statistics clearly show that mandatory programs are much more effective at mitigating seismic risks.

Table 3.8 – California URM Risk Mitigation Statistics by Program Type

After: Paxton, Elwood, Barber & Umland, 2013

	Mandatory	Voluntary	Notification Only	Other	No Program	Total
No. of Jurisdictions	134	39	46	41	23	283
No. of URM Bldgs	19,043	1,269	1,487	3,737	409	25,945
% Retrofitted	70%	16%	7%	15%	4%	55%
% Demolished	17%	8%	6%	11%	27%	15%
% Mitigated	87%	24%	13%	26%	31%	70%

3.3.2 Washington

URM seismic risk mitigation has also been a topic of interest in Washington State, due in large part to property damage and deaths caused by earthquakes in 1949, 1965, and 2001. Efforts in Seattle and Tacoma are reviewed.

3.3.2.1 Seattle

In Seattle, an inventory survey of URM buildings was recently completed by the Department of Planning and Development (City of Seattle 2012). This survey built on past surveys completed for the DPD (Hoover 1992, Reid Middleton 2007). Approximately 800 buildings were identified, 10-15% of which appeared to have received some degree of seismic upgrading.

Recognizing the risks associated with unretrofitted URM buildings, the City of Seattle passed ordinances requiring retrofitting of all URM buildings in 1973; however, the ordinances were repealed a couple of years later due to public opposition and administrative difficulties (FEMA 1998). Currently, comprehensive seismic upgrading is only triggered by changes of use or occupancy, similar to Victoria. However, Section 3401.8 (“Unsafe Building Appendages”) of the Seattle Building Code states that *“Parapet walls, cornices...that are in a deteriorated condition or are otherwise unable to sustain the design loads...are hereby designated as unsafe building appendages”* and *“shall be abated in accordance with Section 102 (City of Seattle 2009).”* This is essentially a narrowing of focus of the general “unsafe condition” clause (in this case Section 102) that is present in most building codes. Unfortunately enforcement of this clause has reportedly been limited (Rogers, et al. 1998).

More recently, development of URM risk mitigation policies has again gained traction in Seattle: recent draft documents indicate that comprehensive mandatory upgrading is being considered, with relaxations to partial upgrading requirements for certain buildings (similar to San Francisco) (City of Seattle 2012b). Table 3.9 shows selected compliance deadlines. Other timelines were set for obtaining an engineering assessment and permit approval. Sanctions for non-compliance may include the following:

- Quarterly fines (\$500 at assessment stage, \$1000 at permit stage, \$45,000 for full compliance deadline)
- Public posting of non-compliance (online or on site)
- Freeze on new permits for the building
- Denial of incentives
- Abatement of the property by the City

Table 3.9 – Proposed Seattle Ordinance Compliance Timelines

Rating Classification	Definition	Apply for Building Permit	Complete Construction
Critical Risk (Highest Priority)	Emergency Services, Shelters, Schools	1 year	7 years
High Risk	4+ storeys on poor soil, 100+ occupants	2 years	10 years
Medium Risk	All others	3 years	13 years

Several incentives were identified by City personnel for consideration, including the following (City of Seattle 2014):

- **Federal grants:** available for public/non-profit owned buildings from the Federal Emergency Management Agency
- **General obligation bonds:** voter-approved municipal bonds for a city-administered retrofit funding program
- **Levies:** voter-approved increase in money collected from each property owner for a City-administered retrofit funding program
- **Tax abatement:** reduction/elimination of property taxes for a designated period of time; owners to use funds for retrofitting
- **Transfer of Development Rights:** allowing owner of buildings in a designated area to sell developable air space above the building to other developers, who could then increase the density of their developments
- **Federal Tax Credit:** 10% (20% for national historic buildings) tax credit for construction costs of seismic retrofits

3.3.2.2 Tacoma

In 1965, just months after Olympia earthquake, the City of Tacoma adopted an ordinance for parapet strengthening. The ordinance specifically identified parapets as hazardous building appendages and made it possible for the city to require abatement (Rogers, et al. 1998). More recently, with the adoption of the International Building Code (ICC 2012) and International Existing Building Code (ICC 2012a), parapet bracing is mandatory when more than 25% of the roof area is re-roofed.

3.3.3 Oregon

In Portland, a publicly available URM survey was completed in the early 1990’s (City of Portland 2001). However, the information has not been verified or updated by the City.

In 2007, a state-wide seismic needs assessment for public buildings was completed (Lewis 2007). This survey included all types of structures, rather than solely URM. Approximately 1800 URM buildings were identified.

Similar to many areas, comprehensive seismic upgrading is currently only required as part of a change of use or occupancy or other significant renovation, although the trigger system in the IEBC (International Existing Building Code) is somewhat detailed. The retrofit rate in Portland is thought to be similar to Seattle (Powell 2011), although statistics are not available. However, the Portland City Code does have a retroactive parapet ordinance requiring an engineering assessment and remediation when roof repairs or replacement are undertaken (City of Portland 2011).

3.4 New Zealand

In New Zealand, the traditional ‘change of occupancy/significant renovation’ triggered seismic upgrades are required. The country also has legislation addressing “earthquake prone” buildings (EPB’s). Note that the definition of an EPB is one that “*will have its ultimate capacity exceeded in a moderate earthquake*” and if it were to collapse “*would be likely to cause injury or death.*” (Government of New Zealand 2014). A moderate earthquake is defined as having one-third the intensity (and same duration) as that specified for new buildings.

Before discussing practices in New Zealand, it should be noted that the nation-wide provisions of the Building Act are under review and could change drastically. Currently, however, each Territorial Authority (a level of local government, sometimes individual cities) is responsible for developing its own EPB policy and is afforded considerable flexibility. Territorial Authorities could either actively identify EPB’s and require strengthening, or identify buildings as owners apply for building consents for other construction work. Note that the method of determining the ultimate capacity is not explicitly specified, but the typical methods used in practice are as specified in New Zealand’s national guidelines for seismic assessment (NZSEE 2006). An “Initial Evaluation Procedure” (IEP) is used for the screening and identification. The IEP for URM buildings is discussed in more detail in Section 6.2.5.

Strengthening is simply required to achieve a capacity such that the building is no longer earthquake-prone (i.e. 34% of current code). However, the New Zealand Society for

Earthquake Engineering (NZSEE) recommends strengthening to 67%NBS⁴ (NZSEE 2006). This viewpoint is relatively consistent with seismic upgrading practices in most countries whereby, once strengthening is triggered, it is customary to strengthen to a higher level than just above the trigger level.

With regard to performance of earthquake-prone buildings and their strengthened counterparts, the Christchurch earthquake of February 2011 provided a significant test. Before discussing the performance it should be noted that the intensity of the earthquake was well in excess of even the design standards in place at the time of the earthquake. Figure 3.1 compares the average response spectra from the earthquake with the design spectra for Christchurch and Victoria.

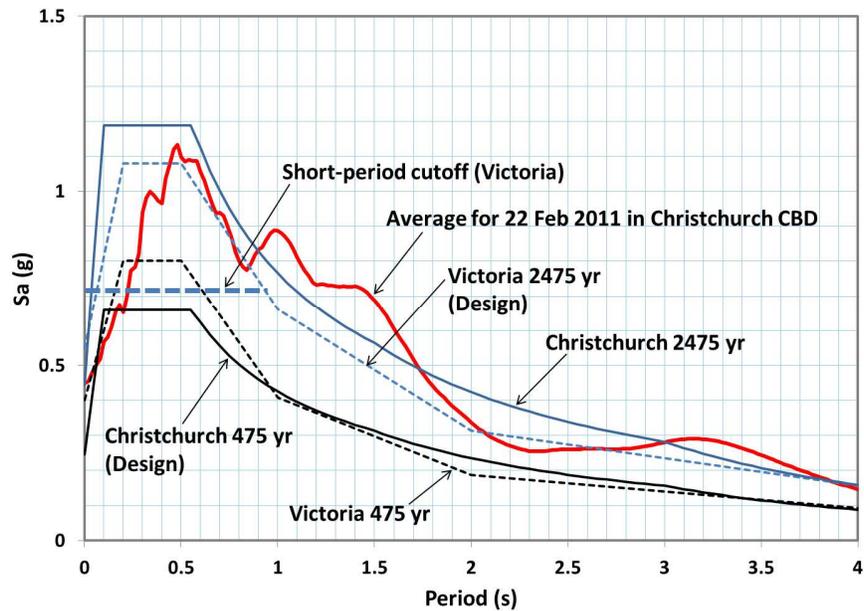


Figure 3.1 – Design Spectra vs. February 2011 Response Spectra

From: Paxton, Elwood, Barber, and Umland, 2013

As expected given the high intensity of shaking, damage was severe. 370 URM buildings in the Christchurch Central Business District (CBD) were surveyed. Of these, the %NBS value was estimated for 125 buildings (Ingham and Griffith 2011b). Figure 3.2 illustrates the observed performance. The results clearly show the trend of improved performance with increased strengthening. Although buildings in the 33%NBS range appeared not to perform particularly well, it should be remembered that the intensity of the earthquake was 150-200%NBS over much of the response spectrum. Buildings strengthened to the 67-100%NBS range performed well.

⁴ “NBS” is an abbreviation for “New Building Standard” (i.e. current code for new buildings)

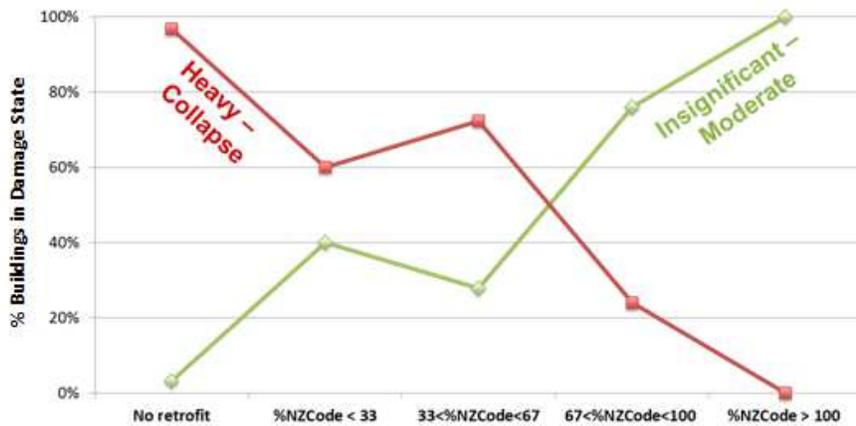


Figure 3.2 – Damage vs. %NBS for 125 URM Buildings in Christchurch
 From: Paxton, Elwood, Barber, and Umland, 2013

3.5 Canada

Having looked abroad for a sampling of seismic risk mitigation practices, the focus will now turn to efforts in Canada. Risk mitigation efforts and seismic upgrading practices are discussed for Vancouver and for Victoria.

3.5.1 Vancouver

The City of Vancouver does not have an inventory of URM buildings. However, it has incorporated a comprehensive section on existing buildings into its building code, the Vancouver Building Bylaw, or VBBL (City of Vancouver 2012): “Part 10 – Existing Buildings” of the VBBL and its Appendix includes triggers clearly indicating when seismic upgrading is required and to what extent. In addition to the traditional triggers of change of use/occupancy and “significant improvements,” Part 10 defines upgrade scenarios triggered by repairs, reconstruction, as well as “minor” and “major” renovations and additions.

For renovations and certain changes to lower risk occupancies, seismic requirements are limited to, at most, securing non-structural items and fall hazards. For major vertical additions (i.e. those that impose additional gravity loads/seismic weight to the existing structure), comprehensive seismic upgrading to at least 75% of current code demands would be required if the subject building’s seismic evaluation shows that the current capacity is less than 60% of current code. There are no statistics available for the number of retrofits achieved under this trigger-based system.

3.5.2 Victoria

Having summarized seismic risk mitigation in many other jurisdictions, we will now focus on Victoria and provide a more detailed account, including the building code, and heritage incentive programs for seismic upgrading.

3.5.2.1 Building Standards

The City of Victoria falls under the requirements of the British Columbia Building Code (BCBC), which is largely based on the model code, the National Building Code of Canada (NRC 2010). Part 1 of Division A specifies the application of the code, including but not limited to, the following activities (as per Clause, 1.1.1.1):

- A change of occupancy of any building
- An alteration of any building (note: an alteration is defined as “*a change or extension to any matter or thing or to any occupancy...*”)
- An addition to any building
- Reconstruction of any building that has been damaged by fire or earthquake or other cause
- The correction of any unsafe condition in or about any building
- An alteration, rehabilitation, or change of occupancy of heritage buildings

Clause 1.1.1.2 (“Application to Existing Buildings”) also specifies the following: “*Where a building is altered, rehabilitated, renovated, or repaired, or there is a change in occupancy, the level of life-safety and building performance shall not be decreased below a level that already exists (see Appendix A).*”

Although the aforementioned Appendix A has no legal effect (as stated in Cl. 1.1.3.1), it contains a variety of relaxations that apply to heritage buildings, mostly concerned with fire safety and accessibility. However it also gives reference to the NBCC 2010 User’s Guide (NRC 2011), Commentary L on application to existing buildings.

Commentary L is relevant to this study because, although it has no legal effect, it prescribes minimum force levels (through reduced load factors) for the evaluation of existing buildings. For earthquake loads, a factor of 0.6 is specified. It goes on to state that the reduced load factors are “*based on maintaining the level of life safety [but not other objectives such as economy] implied by Part 4.*” Finally, it notes that this force level “*should be considered suitable as a triggering criterion for seismic upgrading...*” but that “*...for design of upgrading, the load factor should be increased, preferably to the NBC value.*” For unreinforced masonry buildings, Commentary L also refers to the

NRC “Guidelines for Seismic Evaluation of Existing Buildings,” (NRC 1992) in particular to Appendix A of that document (not to be confused with Appendix A of the building code, as discussed above), which is the Canadian equivalent of the URM provisions in the previously mentioned Uniform Code for Building Conservation, from the United States.

This represents the extent of code provisions and formal guidance on seismic upgrading of existing buildings. As can be seen, the triggers are quite vague and enforcement relies on the judgment of the building officials who enforce the code.

3.5.2.2 Local Enforcement

The aforementioned requirements of the BCBC and recommendations of Commentary L are essentially reflective of the local practice in Victoria for triggering of seismic upgrades: when a building undergoes a change of occupancy or a significant (undefined) renovation, the City requires that a seismic assessment be performed to 60% of current code forces; if the resistance of the structure is less than this threshold, upgrading is required, typically to a level of 60-75% of current code. The exact upgrade force level is not specified, and may vary from building to building. For unreinforced masonry buildings, the issue is somewhat complicated by the fact that the aforementioned Appendix A of the seismic evaluation guidelines was developed to be compatible with the 1990 version of the NBCC and, thus, is not entirely compatible with the current code for new buildings.

Unlike the City of Vancouver, Victoria does not have well-defined formal triggers or target force levels for upgrades and, thus, requirements are essentially left to the discretion of the building officials. However, the intent of the enforcement is similar and Victoria can look to the Vancouver triggers as a guideline. Figure 3.3 provides a summary of the seismic triggering/upgrade practice in Victoria.

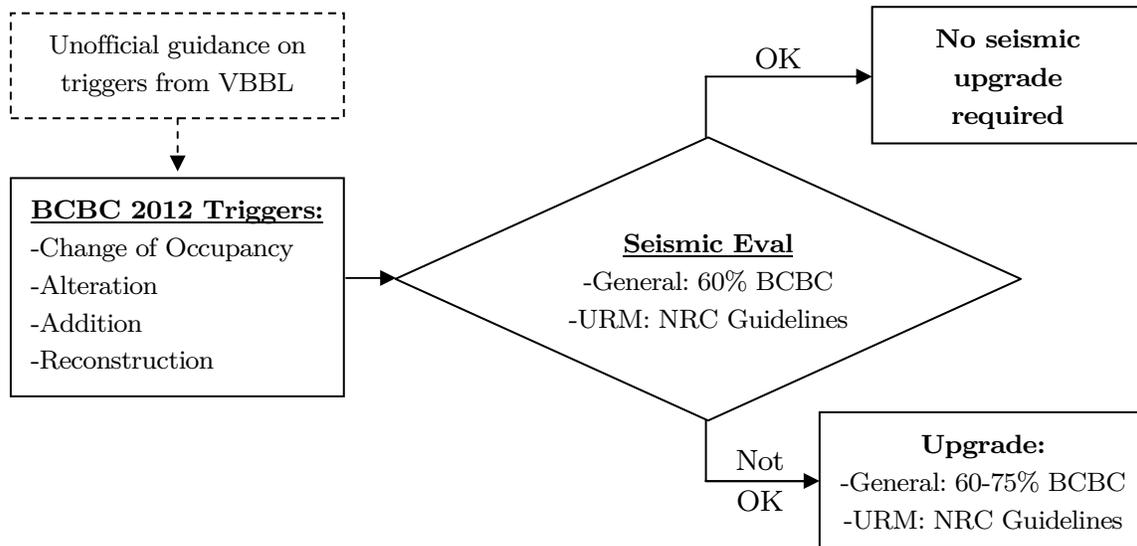


Figure 3.3 – Seismic Evaluation and Retrofit Practices in Victoria

While the aforementioned practice is typical in assessing requirements for overall building seismic upgrades, it is not translated directly in terms of partial upgrades, such as restraint of non-structural components. Seismic restraint is typically required for new buildings and new components of renovations, but there are no triggers for partial upgrading of the remaining components.

As a final note on triggers, Victoria does not make use of the “*correction of any unsafe condition*” clause to require seismic upgrading, even of publicly hazardous non-structural components such as unreinforced masonry parapets and veneers. This is consistent with general practice in the Pacific Northwest. As previously discussed, however, some jurisdictions in the U.S. have adopted mandatory retroactive ordinances requiring bracing of parapets; others have adopted triggered ordinances, whereby the parapets of URM buildings must be assessed (and potentially upgraded) when the building is re-roofed.

3.5.2.3 Heritage Incentives

In the 1970’s, the City began a sophisticated heritage program to conserve its important historic buildings in a core area of downtown known as “Old Town.” Figure 3.4 shows Victoria as well as the extents of Old Town. The city also established an arms-length, non-profit organization known as the Victoria Civic Heritage Trust (VCHT) in 1989 to deliver programs of financial assistance to the owners of protected heritage buildings in the downtown core. The vast majority of these buildings are URM buildings.

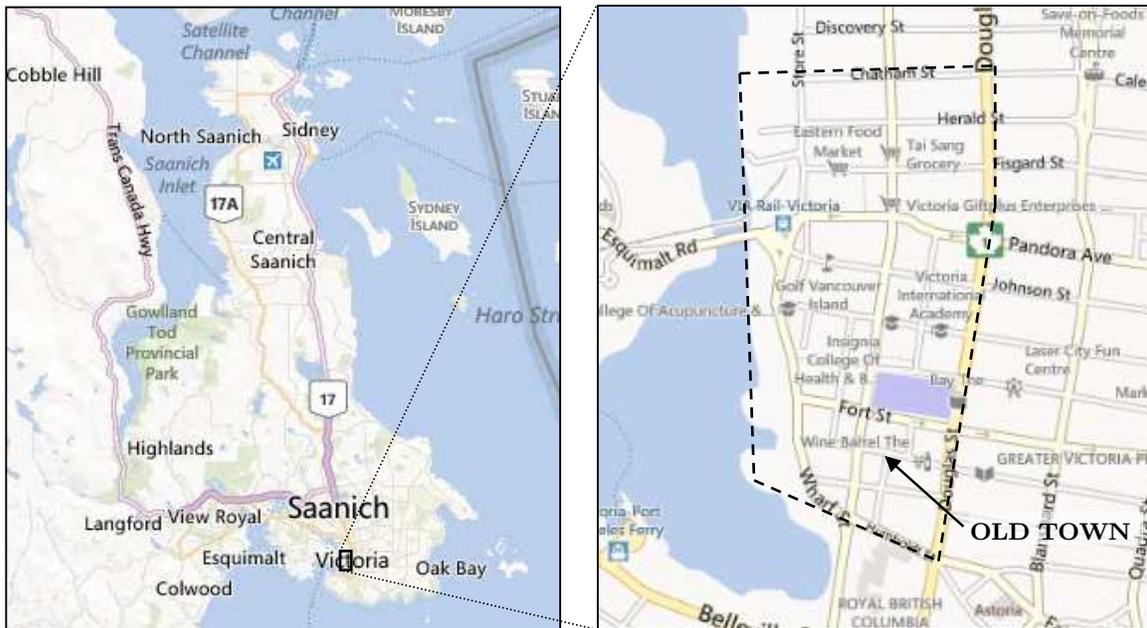


Figure 3.4 – Victoria and ‘Old Town’

After: Paxton, Elwood, Barber, and Umland, 2013

Since 1990, the VCHT has operated the ‘Building Incentive Program’ (BIP), which offers cash grants on a 50% matching fund basis up to \$50,000 to owners of legally protected heritage commercial, industrial, and institutional buildings in Victoria (Victoria 2012a). The purpose of the BIP grant is to promote the conservation and rehabilitation of the buildings; it does not specifically require seismic rehabilitation work, although structural and seismic upgrading work is an eligible cost when allocating grants.

VCHT also assists the City with the administration of the ‘Tax Incentive Program’ (TIP). Started in 1998, this program provides up to a 10-year tax exemption to building owners to assist in the creation of new residential units on the upper floors of underutilized heritage buildings or the substantial renovation of non-residential heritage buildings in the downtown core. The objective is to help offset the high cost of the seismic upgrading. Considered a major success, the program has stimulated the creation of 660 residential units and attracted over \$225 million in private sector investment in 31 seismically upgraded and rehabilitated heritage buildings (City of Victoria 2014). These include former warehouses, industrial buildings, hotels, a department store, and a hospital. Figure 3.5 shows example photos of the types of buildings addressed under the TIP.



Figure 3.5 – Example TIP-Retrofitted Buildings

From: Paxton, Elwood, Barber, and Umland, 2013

Victoria has approximately 200 heritage buildings in the downtown core; this includes both “heritage-designated” buildings (legally protected by municipal bylaws and subject to City Council requirements in the case of alterations) and “heritage-registered” buildings (unprotected, but registered for monitoring and potential future designation) (City of Victoria 2014).

While both the BIP and TIP programs have stimulated a considerable amount of seismic upgrading, a significant challenge for Victoria is the remaining unquantified number (likely a few hundred) of unreinforced masonry buildings which continue to exist with limited occupancy or operate with successful retail businesses on the ground floor and no incentive to undergo a change of use or significant renovation. Provisions for voluntary partial upgrades are likely the only manner in which such buildings will be addressed in a timely manner. Without an inventory of these buildings, the problem cannot be quantified, let alone addressed.

In an effort to promote voluntary seismic upgrading of parapets and facades, the VCHT has developed a new “Parapet Incentive Program” (PIP) for launch in early 2015. The program’s structure is similar to the BIP, except that funds are marked solely for seismic upgrading of parapets and facades (i.e. wall anchorage). The program is currently in a trial phase for a target area. In order to better understand the costs and benefits of such a program, an effort was made in this study to quantify the seismic performance improvement and expected costs/benefits on a probabilistic basis, as presented in Chapter 4 and Chapter 5, respectively.

3.6 Comparing Victoria to Other Jurisdictions

Table 3.10 provides a summary of URM seismic risk mitigation measures throughout the Pacific Northwest. Note that for Seattle the measures have not been finalized, but the intent is to show the general trends in activity. Only selected jurisdictions in California are shown here, but there is a great variety among the 283 affected jurisdictions with URM buildings.

Overall, the efforts to identify and mitigate URM seismic risk in Victoria appear to be lacking in comparison to other regions of the Pacific Northwest. While the incentives for heritage buildings have been successful in promoting re-development, and thus comprehensive seismic upgrading, a complete inventory of URM buildings in Victoria has not been performed and risk mitigation measures are not in place to effectively address occupied buildings, nor are there currently provisions for partial seismic upgrading of buildings to address the most pressing and easily corrected issues of parapet and facade seismic restraint.

Table 3.10 – Comparison of URM Seismic Risk Mitigation by Jurisdiction

Jurisdiction	URM Inventory	Retrofit Requirements	URM Mitigation Rate	Parapet Ordinance
Los Angeles, CA	Yes	Mandatory	88%	Mandatory (enacted 1949)
San Francisco, CA	Yes	Mandatory	86%	Mandatory (enacted 1976)
Palo Alto, CA	Yes	Mandatory	64%	Triggered (Reroofing)
Seattle, WA	Yes	Triggered <i>Proposed Mandatory</i>	10-15%	Mandatory (enacted 1973)
Tacoma, WA	No	Triggered (Detailed System)	Unknown	Triggered (Reroofing)
Portland, OR	Yes	Triggered (Detailed System)	Unknown	Triggered (Reroofing)
Vancouver, BC	No	Triggered (Detailed System)	Unknown	None
Victoria, BC	No	Triggered	Unknown	None

3.7 Additional Considerations

3.7.1 Retrofit Design Standards

Good policy-making is the foundation of a successful URM seismic risk mitigation program. To ensure the retrofits perform as intended, however, appropriate technical standards must be in place. Because the focus of this chapter is not on technical standards or seismic performance, the discussion herein is limited to identification of a few issues that may also come into play in policy making.

Firstly, an appropriate design standard must be specified. In the United States, the International Existing Building Code (ICC 2012a) is the most commonly specified standard. Another possible standard is ASCE 41 (ASCE 2013). Both of these documents are actively maintained with updates published every 3-5 years.

In Canada, the *Guidelines for Seismic Evaluation of Existing Buildings* (NRC 1992) is the most recent and relevant standard. This document was derived from American documents that were the predecessors to the IEBC and ASCE 41. Unfortunately, it is not actively maintained and no updates have been made since its initial publication. Although the NRC guidelines contain much of the content that is still found in the American standards, much new earthquake engineering knowledge has come to light since its publication. For example, the Northridge earthquake showed deficiencies with URM veneer retrofits, mostly due to a lack of proper enforcement and field quality control. The 2010/2011 Canterbury earthquakes showed poor performance of adhesive wall anchors for URM buildings, again largely due to quality control issues. Another issue is that the NRC guidelines were developed to accompany the 1990 NBCC; since then, new editions have been issued in 1995, 2005, and 2010 (with 2015 issues coming soon). Because of significant changes to the NBCC since 1990, the guidelines are not highly compatible with current code for new buildings. This can be problematic because building authorities often specify performance in terms of a fraction of current code (herein abbreviated as “%code”).

3.7.2 Performance-Based Design

An alternative to the aforementioned force-based specification is performance-based design, as provided for in the ASCE 41 standard (recall Figure 2.36). Although a displacement-based approach for a building with largely unknown (and brittle) material properties is likely not as fruitful as it could be for a modern building, this type of standard at least promotes a discussion on performance. On one hand, a retrofit using public money should perhaps use no more resources than are necessary to achieve

minimally acceptable life-safety goals (i.e. “collapse prevention” performance). On the other hand, heritage societies and city planners should be completely aware that retrofits often do nothing to actively ensure that a building will not need to be demolished after a design level earthquake.

3.7.3 Enhanced Design and Construction Supervision

Errors in design (eg. undetected veneer wythes or poor mortar) and quality control issues (eg. adhesive anchor installation) have been the most frequent issues linked to failures of retrofitted URM buildings (Bruneau 1995, Rutherford & Chekene 1997). Moreover, Rutherford and Chekene (1997) showed that the marginal costs for increased design and construction supervision measures are quite small, relative to other improvement measures. Because of these issues, it would seem logical to devote additional resources to enhanced design and construction supervision. Several such positive example programs are in existence. California’s Field Act should certainly be considered successful, as *“not one public school building constructed under the Field Act has collapsed nor has anyone died in earthquakes (CSSC 2007).”*

3.7.4 Other Building Upgrades

Another question to be answered in developing a URM seismic risk mitigation program is the extent to which other building deficiencies, such as accessibility and fire safety should be addressed. Generally, mandatory upgrade programs have excluded accessibility upgrades. In the case of Los Angeles, minimal fire safety upgrades, to the “Dorothy Mae” Ordinance, were completed at a reported cost of \$50,000 per building, on average (Comerio 1989). Other jurisdictions have typically excluded fire safety upgrades.

3.8 Conclusions

Based on the review of California’s statistics, it is concluded that mandatory programs are much more effective at mitigating URM seismic risk than are voluntary (or other passive) programs. However, there are a number of socioeconomic issues to be considered and it is essential that any ordinance must have substantial input from the stakeholders within the community. Based on the facts that Victoria does not have an inventory of URM buildings, does not have ordinances requiring parapet upgrades, and only requires comprehensive upgrades as part of a change of use/occupancy, it is concluded that URM seismic risk mitigation measures in Victoria are lacking compared to other jurisdictions. The same may be said of Vancouver or southwestern BC in general.

Chapter 4

Quantifying Building Vulnerability Through Observed Damage Statistics

4.1 Purpose and Scope

In Chapter 3, URM seismic risk mitigation efforts of several communities were reviewed. It was found that, in most cases, the decision to implement these programs has been based primarily on anecdotal evidence and emotional/political response to disastrous earthquakes. In order to develop a more rational basis for such decisions, it is first necessary to quantify the seismic performance of such buildings in a systematic way. The improvements can then be weighed against the costs of strengthening and a decision made regarding the appropriate action.

In this chapter, a statistical basis for the performance of unreinforced masonry buildings is developed, including the effects of strengthening and various base building characteristics. These results will subsequently be used in Chapter 5 to perform a cost-benefit analysis for URM seismic strengthening and in Chapter 6 as a basis for developing a seismic screening system for URM buildings.

4.2 The Role of Damage Observations in Engineering and Policy-Making

Observations from earthquakes have long played an important role in the advancement of earthquake engineering and in policymaking. The 1906 San Francisco earthquake led to the formation of the Structural Association of San Francisco, which later became the Structural Engineers Association of California (MCEER 2008). In response to the 1925 Santa Barbara earthquake, the first seismic provisions in the United States were published in the 1927 Uniform Building Code (Risk Management Solutions 2006). The 1933 Long Beach earthquake resulted in immediate passage of the Field Act, which prohibited the use of unreinforced masonry construction for new schools (Rutherford & Chekene 1997) and effectively marked the end of unreinforced masonry construction in many locations through changes to building codes (Risk Management Solutions 2006). Another significant outcome of the Long Beach earthquake was the first comprehensive, systematic damage survey in the United States, including damage scales for various building types and statistical break downs (Wailles and Horner 1933).

Reconnaissance by researchers, building officials, and engineers from past earthquakes has provided a wealth of information on the seismic performance of unreinforced masonry buildings (and structures in general) and led to many important developments. However, much of the reconnaissance has been anecdotal in nature. Fortunately, systematic damage surveys and statistical analyses of performance have become more common over the past few decades, particularly with the increasing availability of strong motion recordings throughout the late 1980's and 1990's. Such studies have been invaluable because they have permitted the quantification of performance results and, ultimately, more rational and objective conclusions about seismic performance and risk mitigation alternatives.

In this chapter various damage assessment methodologies encountered in this study are reviewed, as are the various sources of damage statistics. Subsequently, some of the more pertinent unreinforced masonry damage surveys in the literature are presented and, finally, further work by the author using various existing databases is presented. As will be seen, a reasonably substantial body of work on this topic was completed throughout the 1980's and 1990's. The impetus for the work stemmed largely from the URM seismic risk mitigation programs that were being enacted throughout California at the time, such as the 1986 "URM law" (California Legislature 1986) and Los Angeles' Division 88 (City of Los Angeles 1985). Building owners, engineers, and city officials alike were interested in quantifying the benefits achieved through these expensive mitigation strategies. Since this time, however, interest has waned as the affected communities have completed their mitigation programs. Little information has been collected on URM buildings in British Columbia. Therefore, a thorough review of existing information is needed to capture the range of possible performance of British Columbia URM buildings. The collection presented herein is certainly not exhaustive, but is believed to represent a sample that sufficiently supports the conclusions reached.

4.3 Review of Systematic Damage Assessment Methods

As might be expected based on the number of different interested parties and possible uses of the results, there are many different damage assessment methods and/or rating scales. The relevant scales for this study include those from Wailes and Horner (1933), ATC-13 (ATC 1985), ATC-20 (ATC 1989), and HAZUS (FEMA 2012). There are several other similar damage scales in the literature, but the focus herein is on those which see use in the damage surveys encountered in this study. In all cases, the damage scales separate the continuum of possible seismic damage into discrete "damage states" which are linked to tangible damage descriptions or repair needs (eg. "*Parapets fell,*

separation of veneer from backing” or “extensive structural damage requiring repair...”). In some cases, the damage states are also related to loss values.

4.3.1 Wailes and Horner

As aforementioned, this scale was developed in conjunction with an assessment of damage from the 1933 Long Beach earthquake. The scale uses five damage states (A through E), each associated with a description and example photograph for each type of structure. Table 4.1 provides the damage state descriptions and illustrative photos for URM buildings. Note that available reproductions of the original photos are low-quality and so more recent, higher-quality photos are provided. The photos are from various sources, as noted, and were selected to be similar to the original photos. Rutherford & Chekene (1997) provides the original photos.

4.3.2 ATC-13

The ATC-13 report was developed by the Applied Technology Council and published in 1985 (ATC 1985). The main purpose behind the study was to develop a loss estimation methodology that could be applied on a regional scale throughout California. Before discussing its use in damage surveys, we will discuss its background and theory as a loss estimation tool to better understand what the collected damage statistics should represent. This is a worthwhile endeavour, since the majority of the damage statistics in this study use the ATC-13 damage scale.

ATC-13 was considered a landmark study in earthquake loss estimation and much research effort was subsequently expended throughout North America to adapt it to other regions (Blanquera 1999, Onur 2001, Thibert 2008). Until approximately the year 2000, ATC-13 was the most commonly used methodology in North America. Even today, its roots are embedded in newer methodologies, such as HAZUS (FEMA 2012), which have become more popular. The main difference between the ATC-13 and HAZUS methodologies is that ATC-13 was based primarily on expert opinion, while HAZUS takes a more analytical approach and is based on nonlinear static analysis of prototypical buildings.

The ATC-13 methodology accounts for various forms of losses, including:

- 1) Direct losses (losses due to damage caused by earthquake effects, eg. repair costs)
- 2) Indirect losses (loss of functionality, eg. downtime)
- 3) Social losses (eg. casualties)

Table 4.1 – Wailes and Horner Damage Scale

Damage State	Description
A	Undamaged or small cracks only (from Ingham & Griffith, 2011) 
B	Parapets fell, separation of veneer from backing (from Ingham & Griffith, 2011) 
C	Major damage to less than 50% of bearing walls (http://reidmiddleton.wordpress.com) 
D	Major damage to more than 50% of bearing walls (from Ingham & Griffith, 2011) 
E	Total loss; collapse or severe damage requiring demolition (http://www.newswire.co.nz/2011/02/day-4-the-search-for-survivors-continues/) 

The losses are predicted as a function of the earthquake intensity (i.e. the hazard) and the expected response of the building to these demands (i.e. the vulnerability). Therefore, the methodology is a form of risk analysis. As will be seen herein, compiling and interpreting earthquake damage statistics is a different but related problem.

At the time of the development of ATC-13, ground motion recordings were much more scarce than today. Due to this lack of availability of quantitative engineering measurements of earthquake intensity in real earthquakes, it was decided to use the Modified Mercalli Intensity (MMI) scale as the measure of earthquake intensity (ATC 1985). MMI contour maps have long been routinely developed by geological/seismological agencies such as the United States Geological Survey based on “felt reports” from earthquakes (i.e. witness accounts of shaking and damage). Details on the MMI scale are commonly available in earthquake engineering literature and are not presented here. Various building prototypes (78 in total for ATC-13) are defined to represent the vulnerability. The classifications are general in nature (eg. “High-Rise Concrete Shear Wall Building” or “Low-Rise Unreinforced Masonry Building”) and serve as a baseline indicator of the expected losses for a given building; the focus of the classification is on the seismic force resisting system. Unfortunately, these classifications do not account for several particularities of structures such as irregularities of the seismic force resisting system or the era of design and construction. While these classifications have proven effective on a regional scale, a more detailed assessment is often warranted when examining a specific subset of the building stock, such as strictly unreinforced masonry buildings. In ATC-13, damage is discretized into seven Damage States ranging from “None” to “Destroyed.” Each damage state is associated with a given damage factor⁵ range (Damage Factor = Repair Cost/Building Replacement Value) and a Central Damage Factor (CDF), which is usually taken as the arithmetic mean Damage Factor for that range, although some have instead advocated the use of the geometric mean (Wiggins, Breall and Reitherman 1994). Results are presented in what are called Damage Probability Matrices (DPMs). Table 4.2 provides the damage states and Table 4.3 is an example of such a DPM.

This procedure also lends well to processing observed statistics. However, in ATC-13 the DPMs were developed using expert opinion: more specifically the Delphi survey method was employed. This method was employed due to a lack of observational data and is discussed in detail in the original ATC-13 document. Since this time much data has

⁵ The terms “damage ratio” and “damage factor” are both used in the literature. In this thesis, they can be taken as synonyms

become available and DPMs based on observed data have been produced in the literature, such as will be reviewed and undertaken herein.

Table 4.2 – ATC-13 Damage States

Damage State		Description	Damage Factor Range	Central Damage Factor
1	None	No Damage	0%	0.0
2	Slight	Limited localized minor damage not requiring repair	0-1%	0.5%
3	Light	Significant localized damage of some components generally not requiring repair	1-10%	5.5%
4	Moderate	Significant localized damage of many components warranting repair	10-30%	20%
5	Heavy	Extensive damage requiring major repairs	30-60%	45%
6	Major	Major widespread damage that may result in the facility being razed, demolished, or repaired	60-100%	80%
7	Destroyed	Total destruction of the majority of the facility	100%	100%

Table 4.3 – Example Damage Probability Matrix

(Modified from ATC-13)

Damage State		Damage Factor Range	CDF	Probability of Being in Damage State For A Given MMI [%]						
				VI	VII	VIII	IX	X	XI	XII
1	None	0%	0.0	95	49	30	14	3	1	0.4
2	Slight	0-1%	0.5%	3	38	40	30	10	3	0.6
3	Light	1-10%	5.5%	1.5	8	16	24	30	10	1
4	Moderate	10-30%	20%	0.4	2	8	16	26	30	3
5	Heavy	30-60%	45%	0.1	1.5	3	10	18	30	18
6	Major	60-100%	80%	-	1	2	4	10	18	39
7	Destroyed	100%	100%	-	0.5	1	2	3	8	38
MDF = $\Sigma[P(DS_i)*CDF_i]$ =				0.2%	2.9%	6.6%	14.3%	25.9%	42.4%	78.0%

In examining Table 4.2 it can be seen that the Damage Factor Range increases with each damage state, which represents the increasing uncertainty in the losses as damage increases. For instance, the range of possible repair costs for a building with only “significant localized damaged, generally not requiring repair” must be small (for example some painting or plaster repairs). Conversely if “extensive damage requiring major repairs” is present, then the scope of potential repair work is much more unclear.

In examining Table 4.3 it is worth noting that the DPM provides a framework that explicitly considers the uncertainty in the relationship between ground motion intensity (such as MMI) and damage. For example, it can be seen that if a subject building experiences ground shaking of intensity of MMI VIII, there is a 40% probability that the building will be in the “Slight” damage state, a 16% chance of being in a “Light” damage state, and so on. Another way of expressing the probability would be to say that there is an 86% probability that the building will fall into a damage state of “Light” or lower (i.e. 40%+30%+16%). One can also readily compute the standard deviation of the MDF and thus establish reasonable upper and lower bounds for the expected damage.

4.3.3 ATC-20

ATC-20 (ATC 1989) is a post-earthquake safety evaluation method commonly employed by communities throughout North America and abroad. The methodology is designed to facilitate rapid assessment of the safety of buildings for occupants and passersby. It involves a somewhat cursory review of the building, which may or may not involve interior access. The primary end result is a ‘placard’ that is posted at the building, indicating the level of permitted access. Three different placards are possible, as shown in Table 4.4.

Although the methodology is not specifically intended for damage assessment, the safety of a building for occupancy is obviously closely related to the damage and many researchers have used tagging data in damage surveys. Section 4.4.1 provides a discussion of the associated merits and drawbacks of this form of data.

Table 4.4 – ATC-20 Placards

Placard	Description
<p>Green “Inspected”</p>	<p>The building was reviewed and no access restrictions were imposed</p>  <p>The image shows a green placard with the following text: INSPECTED, LAWFUL OCCUPANCY PERMITTED. It includes fields for Date, Time, and Inspector ID / Agency. It also has checkboxes for 'Inspected Exterior Only' and 'Inspected Exterior and Interior'. A note states: 'Report any unsafe conditions to local authorities; reinspection may be required.' At the bottom, it says 'Do Not Remove, Alter, or Cover this Placard until Authorized by Governing Authority.'</p>
<p>Yellow “Restricted Use”</p>	<p>Access is only for possession retrieval and access to portions of the building may be prohibited</p>  <p>The image shows a yellow placard with the following text: RESTRICTED USE. It includes fields for Date, Time, and Inspector ID / Agency. A note states: 'Caution: This structure has been inspected and found to be damaged as described below.' Another note says: 'Caution: Aftershocks since inspection may increase damage and risk.' At the bottom, it says 'Do Not Remove, Alter, or Cover this Placard until Authorized by Governing Authority.'</p>
<p>Red “Unsafe”</p>	<p>No access is permitted</p>  <p>The image shows a red placard with the following text: UNSAFE, DO NOT ENTER OR OCCUPY (THIS PLACARD IS NOT A DEMOLITION ORDER). It includes fields for Date, Time, and Inspector ID / Agency. A note states: 'This structure has been inspected, found to be structurally damaged and is unsafe to occupy, in accordance with...'. Another note says: 'Do not enter, except as specifically authorized or written by jurisdiction. Entry may result in death or injury.' At the bottom, it says 'Do Not Remove, Alter, or Cover this Placard until Authorized by Governing Authority.'</p>

4.4 Collecting Damage Data

As previously noted, the most useful form of damage survey is one in which observations are systematically collected and, thus, the observed performance can be objectively quantified and statistically analyzed for various hypotheses of interest, which entails going beyond the commonly completed reconnaissance papers with case studies or anecdotal evidence. It entails collecting a representative sample of all structures for the given population of interest (in this case, unreinforced masonry bearing wall buildings), not just those that performed noticeably well or poorly. As a preface to the damage statistics presented later in this chapter, it is worth discussing the various sources.

Damage statistics are typically collected in two types of surveys:

- Post-earthquake safety evaluations performed for the local building authority
- Ad-hoc surveys for research purposes

4.4.1 Statistics from Post-Earthquake Safety Evaluations

Damage statistics are often collected during post-earthquake safety evaluations, but the primary focus in this case is the safety of potential building occupants and passersby. There are often two outcomes: an ATC-20 placard indicating the level of access permitted for the building and a damage percentage, representing the cost of repairs as a fraction of the building replacement value. The ATC-13 damage states (see Section 4.3.2) are quite commonly used to describe the anticipated cost of repairs. However, there is significant uncertainty in the results for the following reasons:

- 1) The background and experience of those completing the assessments is variable. Therefore, the accuracy of any individual assessment could be affected
- 2) Many assessors are usually involved. Therefore there can be inconsistencies in the interpretations of damage. As noted by Lizundia (1993), *“one assessor may look at a cracked parapet and call it a 1% loss, assuming that repointing the mortar cracks would be necessary. Another may see the same cracks, imagine that a complete parapet retrofit will be required, and call it a 10% loss”*
- 3) The level of detail in the assessment can vary. For example, the ATC-20 methodology offers both “Rapid” and “Detailed” assessments. In the former of the two, the building is generally not entered. Placard statistics for URM buildings in the Northridge earthquake for initial inspections vs. re-inspections are compared in Figure 4.1 (Rutherford & Chekene 1997). Obviously a rapid assessment (from the exterior) would produce less reliable estimates of the damage, as the interior structural and nonstructural components are not inspected

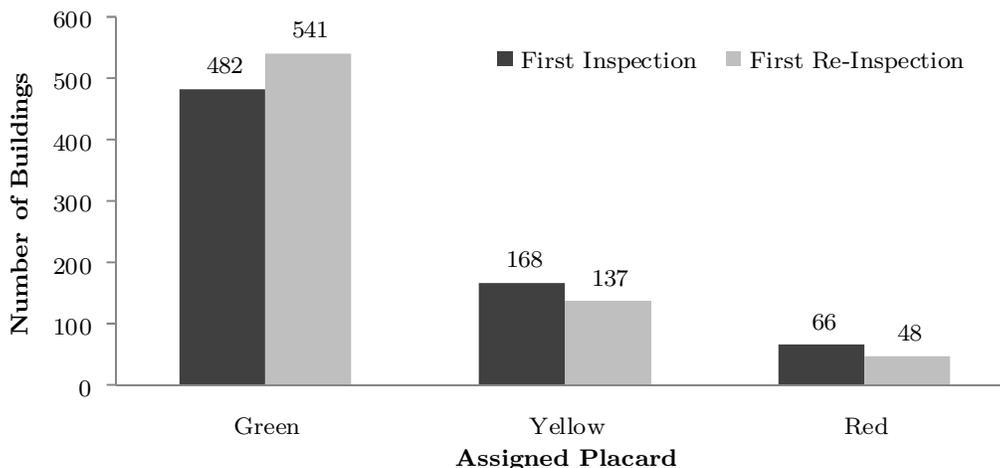


Figure 4.1 – URM Placard Statistics Comparison from Northridge Earthquake

(Reproduced From Rutherford & Chekene, 1997)

- 4) There can be sampling bias, because buildings that are lightly damaged are less likely to be assessed

4.4.2 Statistics from Ad-Hoc Damage Surveys

Academics, building officials, and practicing engineers may perform damage surveys specifically for the purpose of compiling damage statistics and quantifying seismic performance/losses. These studies often are more detailed, but may not cover as large a fraction of the total building population as resources can be more limited and the assessments are more in-depth. A related merit is that there is less chance of inconsistencies between assessors, as there are often fewer individuals and greater communication between them. The level of detail in these studies can also vary, yielding significantly different results. Figure 4.2 compares damage ratios for two different surveys from the 1983 Coalinga earthquake: as noted by Lizundia (1993), the study by Shah et al. (1984) consisted of a one-day walk-through by a team of engineering students, while a study by Reitherman et al. (1984) was sponsored by the National Science Foundation and included a walk-through, materials testing, a photo review process, and discussion between at least two engineers before a damage level was assigned. As shown in the figure, the mean damage ratio of the former was 50%, compared to 35% for the latter.

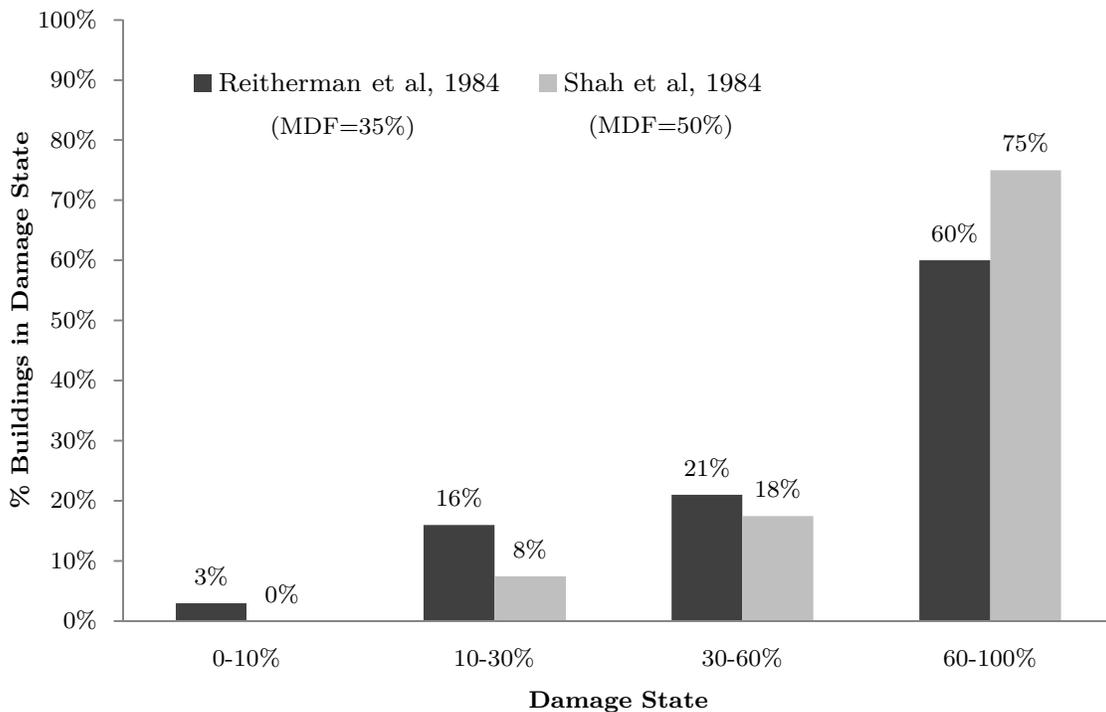


Figure 4.2 – Damage Ratio Comparison from Coalinga Earthquake
(Reproduced From Lizundia, 1993)

It should be noted that some researchers (Ingham and Griffith 2011b) have combined the “none” and “slight” damage states in their surveys, as it may be difficult to ascertain that there is indeed no damage whatsoever. At the opposite end of the damage scale, one could argue that it is also difficult to distinguish between the “Major” and “Destroyed” damage states. One case in point is noted by Lizundia (1991): in this damage survey of URM buildings in the Loma Prieta earthquake of 1989, a portion of the data came from post-earthquake safety evaluations completed by San Francisco building officials. 16 of 1923 unretrofitted URM buildings were originally assigned damage states of “Major” or “Destroyed” (exact figures not disclosed) but as noted by Lizundia *“the follow-up survey indicated that none of these 16 buildings were actually destroyed and a better estimate of the damage they suffered [was] approximately 60% of their replacement costs.”* Such sources of error and uncertainty in assigning damage states is best addressed by careful re-examination of the data and, when possible, comparing the results of more than one damage rating method.

There is clearly substantial uncertainty regardless of the chosen evaluation method. Nonetheless damage statistics play an important role in assessing and improving earthquake performance of structures, as they can account for effects not adequately captured by analytical models.

4.5 Ground Motion Intensity Measurements

With the damage data collected, one could simply compile overall damage statistics for the entire sample and get a sense of how the structures performed. However, results are often much more useful when sorted by the intensity of seismic demand that each specific building experienced. Since ground motion is only measured at a few select locations throughout the region, one must estimate the ground motion experienced at each site. This section discusses the various ground motion intensity measurements (IM’s) that were used in this study and manners of estimating an IM at a given site.

4.5.1 Intensity Measurements

Identifying ground motion parameters that are good indicators of damage has itself received a fair amount of attention in the literature. This section discusses the IM’s that were encountered in this study.

4.5.1.1 Modified Mercalli Intensity

The Modified Mercalli Intensity (MMI) scale was traditionally used as the intensity measurement in compiling damage statistics, as it was the only commonly available

measurement (ATC 1985). The validity of MMI as an indicator of damage has been questioned by some (Rutherford & Chekene 1997, Onur 2001, Thibert 2008). The basis of this criticism is that MMI – which is partially measured in terms of damage (eg. “...parapets will topple...” – is used as a predictor of damage. Another issue is that the MMI scale perhaps does not provide enough “resolution” in the demand: for example, MMI IX is typically associated with peak ground accelerations of 0.65-1.24g (USGS 2011). If a jump in damage occurs in this range, there is significant uncertainty as to the seismic demand at which the jump actually occurred.

More recently the Instrumental Intensity (I_{MM}) scale was developed by Wald et al. (1999) for automated generation of shakemaps by the United States Geological Survey. The scale was developed through a regression analysis of MMI versus PGA and PGV for several California earthquakes, including all of those discussed in this report. The regression equations are as follows:

$$I_{MM}=3.66*\log(PGA) - 1.66, I_{MM} \leq 7 \quad (4-1)$$

$$I_{MM}=3.47*\log(PGV) + 2.35, I_{MM} > 7 \quad (4-2)$$

Although the use of I_{MM} offers a somewhat less subjective measurement, it must be remembered that it is still closely related to the observed damage (especially for the subject earthquakes that were included in the regression analysis) and thus the circularity issue would still apply when used as an intensity measure for damage surveys.

4.5.1.2 Peak Ground Response Values

The use of peak ground acceleration (PGA) has been investigated in some studies relating damage to ground motion (Lizundia, Dong and Holmes 1993, Rutherford & Chekene 1997, King, et al. 2005). The merits of using PGA include its simplicity, lack of period-dependence, and the fact that associated hazard values are readily available; this latter item is important when the results are to be used in loss estimates, as is the case in this study. However it has drawbacks in that PGA is not, strictly speaking, associated with soils amplification factors (NEHRP 1994). Applying short period amplification factors seems reasonable and has been performed in some cases, such as HAZUS (FEMA 2012). Another drawback is that PGA is sensitive to high-frequency ground motions, which may not impact the structure significantly (King, et al. 2005).

Peak ground velocity (PGV) has been investigated by others (Lizundia, Dong and Holmes 1993, King, et al. 2005) and was shown to be a reasonably good indicator of

damage. Again, the fact that PGV is not period-dependent would be considered a merit. However, most modern seismic hazard analyses do not include PGV values and, thus, it would be necessary to first perform a seismic hazard analysis before applying the results to a loss estimate.

4.5.1.3 Spectral Response Values

The use of an elastic spectral response value has merit in that it can represent more accurately the response of the building. The use of five percent (5%) damped spectra is typical. Both spectral acceleration (S_a) and spectral velocity (S_v) have been shown to correlate reasonably well with damage (Lizundia, Dong and Holmes 1993, King, et al. 2005). Spectral displacement (S_d) has also been shown to be a good indicator of damage and is the primary intensity measurement in HAZUS (FEMA 2012). Of course, the period-dependence can be a potential drawback if an inappropriate period is selected and there is a large spike at this point of the response spectrum for the subject ground motion. Lizundia et al. (1993) made use of a building code period expression in an attempt to address the issue of period selection. Although some improved correlation was noted, the expression used was based on the number of storeys, which is fundamentally flawed for typical URM buildings with flexible diaphragms. Moreover, many buildings may not be adequately connected so as to respond as a single unit. Using multiple periods (i.e. different periods for different buildings) also presents a complication when performing loss estimates.

Given the aforementioned issues and the considerable uncertainty in the remainder of the process, it is the author's opinion that the choice of period for spectral values should be governed largely by simplicity. For example, representative short ($T=0.2\text{sec}$) or long ($T=1.0\text{sec}$) periods could be used, both of which can be directly associated with seismic hazard values that underlie current Canadian codes (Adams and Halchuk 2003). The use of a readily available spectral value eliminates the added effort of performing a probabilistic seismic hazard analysis.

Of the two candidate periods, $S_a(1\text{sec})$ is thought to be the more representative of URM buildings and is commonly used in a variety of URM seismic assessment procedures of both diaphragms (ICC 2012a) and out-of-plane wall response (Derakhshan 2011, ICC 2012a, Penner 2013). The more pronounced effect of soils on $S_a(1)$ than $S_a(0.2)$ is also consistent with the observed profound effect of soils on the performance of URM buildings, as will be subsequently demonstrated in this chapter. Another reason for using $S_a(1)$ instead of $S_a(0.2)$ (or PGA) is the reduced spatial variability. Figure 4.3 from

Turner et al. (2010) provides an example plot of short period (0.2 second) and long period (1.0 second) spectral accelerations for recorded values versus values at a distance of 500 feet. This assumes similar soil conditions, but there is already significant dispersion, particularly for the short period.

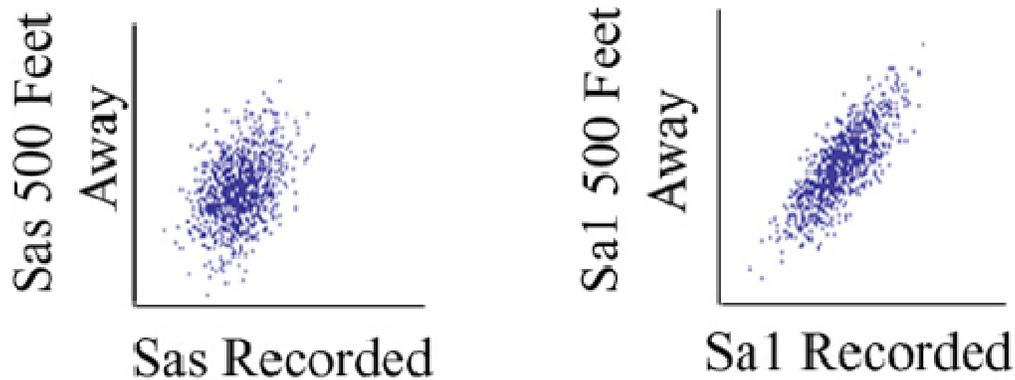


Figure 4.3 – Spatial Variation in Spectral Acceleration

(From Turner et al., 2010)

4.5.1.4 Cumulative Absolute Velocity

Cumulative absolute velocity (CAV) is defined as the integral of the absolute value of the acceleration time history, as shown below:

$$CAV = \int |a(t)| dt \quad (4-3)$$

Its use as a damage indicator has been investigated in various studies (Cabanas, Benito and Herraiz 1997, Moon, et al. 2014), with good results. Typically, the above definition is modified to include some threshold value of acceleration, below which there is no contribution to damage. The motivation for the use of CAV is that it captures intensity and duration, as well as a critical value of acceleration below which there is no effect. In the case of multiple earthquakes, it can be used to capture the cumulative ground shaking.

While CAV represents a fairly robust and accurate IM, it has drawbacks in its ease of use. There is much less literature regarding CAV, including ground motion prediction equations and the effects of soils. It is also not a meaningful parameter in terms of structural design and cannot be readily communicated to decision-makers in terms relative to, for example, current seismic design standards.

4.5.1.5 Selection of Intensity Measure for This Study

As can be seen, there is a great variety of intensity measures. Those mentioned here provide only those most relevant to this study. The aforementioned works by Lizundia et al. (1993) and King et al. (2005) provide a more in-depth treatment of the subject. For the purposes of this study, a single intensity measure must be selected as the basis for all damage statistics and loss estimates.

MMI or instrumental intensity has a long history of use in damage surveys, but would require a seismic hazard analysis for application in loss estimates; there is also the aforementioned issue of circularity in using MMI as an indicator of damage. For these reasons, it was decided to not use MMI or I_{MM} as the intensity measure.

CAV is the most appealing from theoretical and forensic standpoints, but introduces significant additional effort (and uncertainty). While it is certainly possible to overcome the aforementioned difficulties in using CAV, the purpose of this study was not to develop a motion-damage relationship using the best possible IM, but rather to use the results of a reasonable motion-damage relationship to address a broader host of issues, such as quantifying the economic and societal impacts of URM risk mitigation. Due to the breadth of topics addressed in this study, it was decided to not pursue using CAV as the intensity measure.

Peak ground acceleration and spectral acceleration values offer the greatest ease of use, since they are routinely mapped for earthquakes. The intermediate results of this study (eg. fragility curves) will also be most useful to others if they are in terms of one of these parameters because such values can be readily extracted from building codes. Of these two, spectral acceleration is thought to be more representative of building response and damage. It is noted that Penner (2013) found $S_a(1)$ to be the preferred IM for URM out-of-plane assessment. Ultimately, it was decided to use spectral acceleration at a period of one second, $S_a(1)$, as the intensity measure for use in this study.

4.5.2 Estimating the Intensity Measurement at a Site

Having selected a given intensity measure, a value must somehow be assigned to each building. Several methods have been employed in the literature, including the following:

- Closest station
- Interpolation between stations
- Statistically-derived values

4.5.2.1 Closest Station

If the recorded ground motion is sufficiently close to the building, it can be assumed that the ground motion is the same at both locations. However, the distance must be quite small: a limit of 1000 feet has been used in past studies (ATC 2001, King, et al. 2005). While this method is convenient to implement, it potentially necessitates the exclusion of many buildings from an overall sample. For example, King et al. made use of the Rutherford and Chekene (1997) damage statistics for rehabilitated URM buildings in the Northridge earthquake, but King et al. reported that only 50 of about 800 buildings qualified for use in the study. This is clearly not a sufficient sample size to produce statistically meaningful results, especially after filtering for various criteria (eg. type of retrofit, storey height, etc.). The use of closest station values are likely more appropriate for a building-specific forensic-type study.

4.5.2.2 Interpolation Between Stations

Interpolation between recording stations has been employed in some studies (Lizundia, Dong and Holmes 1993, Rutherford & Chekene 1997). Of course, one could question the accuracy of the interpolation on the basis of the earlier discussion of spatial variability; this would likely be a crucial issue in a forensic-type study of a specific building, but it should be recalled that in this study, several hundred buildings are typically included in a bin and the results focus primarily on the mean value which is captured quite well given the high number of samples.

Furthermore, in an effort to address issues with spatial variability, soil conditions, and proximity to the epicenter, Lizundia et al. (1993) took the following steps:

- Interpolation between stations and buildings using ground motion prediction equations rather than, for example, linear interpolation
- Interpolation only between stations with soil conditions similar to the buildings
- Weighted interpolation between stations based on proximity (for example Lizundia weighted the values based on the square of the distances)

4.5.2.3 Statistically-Derived Values

A more rigorous method of “interpolating” is to derive values based on conditional probability theory. This process combines theoretical values (based on ground motion prediction equations) with the measured values to generate expected values (and, if desired, a distribution) for the ground motion parameter in question. In short, the method yields results that approach the measured ground motion parameter at the stations and that approach the calculated value as the distance from stations increases.

The basis for the gradient is the spatial correlation as a function of distance. Figure 4.4 shows such a function from the work of Goda and Hong (2008). Bradley and Hughes (2013) outline the theory in detail and completed such an exercise for the Peak Ground Accelerations during the various earthquakes of the Canterbury earthquake sequence. Figure 4.5 reproduces the map for the February earthquake from the original paper.

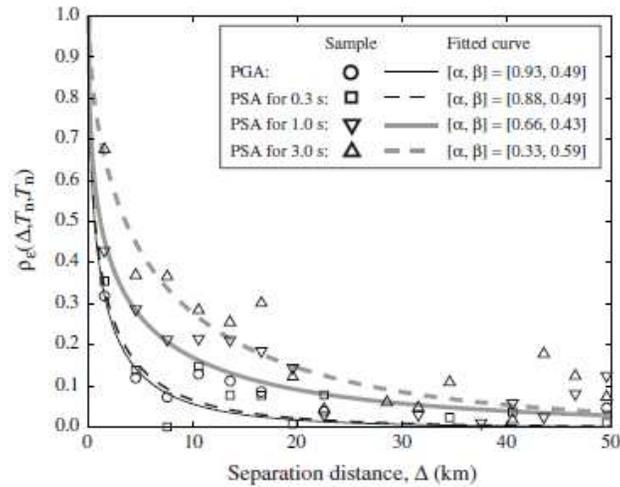


Figure 4.4 – Spatial Correlation Between Ground Motion Parameters
(From: Goda and Hong 2008)

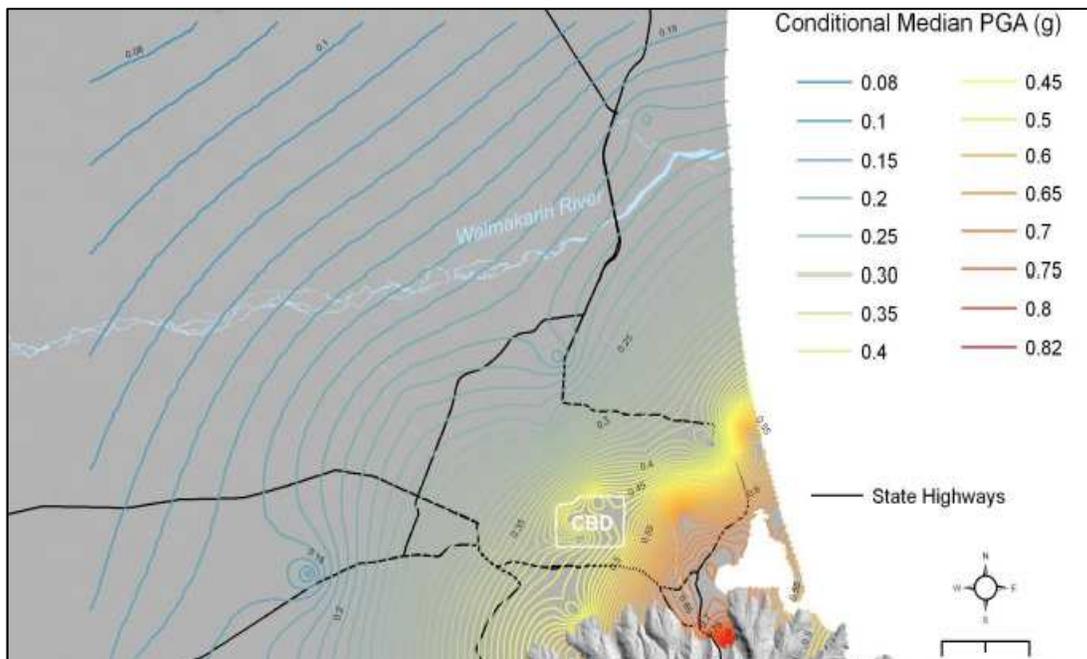


Figure 4.5 – Map of Statistically-Derived PGA for Christchurch Earthquake
(From: Bradley and Hughes 2013)

This is obviously a more scientifically justified method of estimating the IM at the subject buildings than the interpolation method described in the previous section. However, one key weakness of this method is that a single site class must be assumed in the ground motion prediction equation. Depending on the regional geology, this may or may not be a reasonable limitation.

4.5.2.4 Selection of Method for Estimating IM at Sites in This Study

Similar to the choice of IM, it was desired to select a method of estimating that would be reasonably simple for all the available data and yet sufficiently accurate so as not to invalidate the conclusions reached (e.g. whether or not certain retrofit measures are economically justified for various stakeholders). To this end, the results of the three aforementioned methods were investigated for the Canterbury database (as presented in Section 4.7.4 and 4.8.4). Figures 4.6 and 4.7 provide histograms of the number of buildings versus PGA. Note that in Figure 4.7, some bins have been aggregated and others excluded where there are insufficient samples to generate the damage probability matrices and motion-damage relationships (as presented in Section 4.8). Note that even for the first histogram, the IM's estimated at each site have been rounded to the nearest .05g.

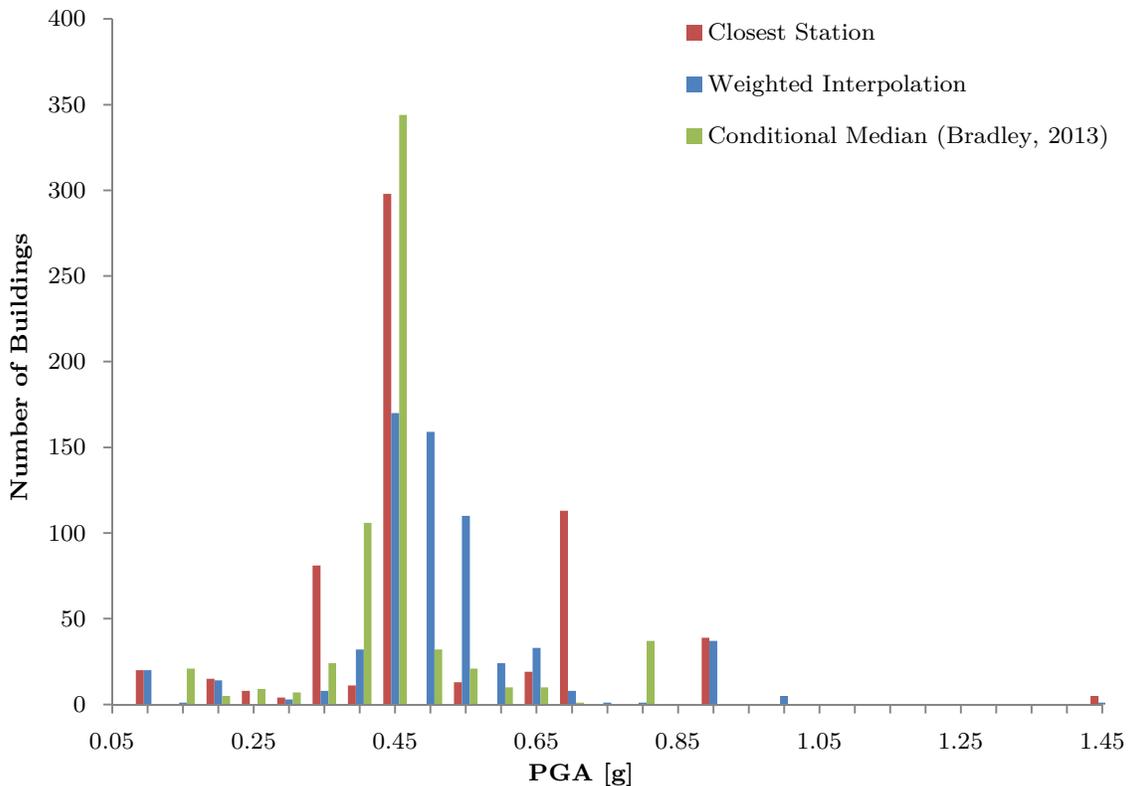


Figure 4.6 – Number of Buildings vs. PGA for Canterbury Database (Initial)

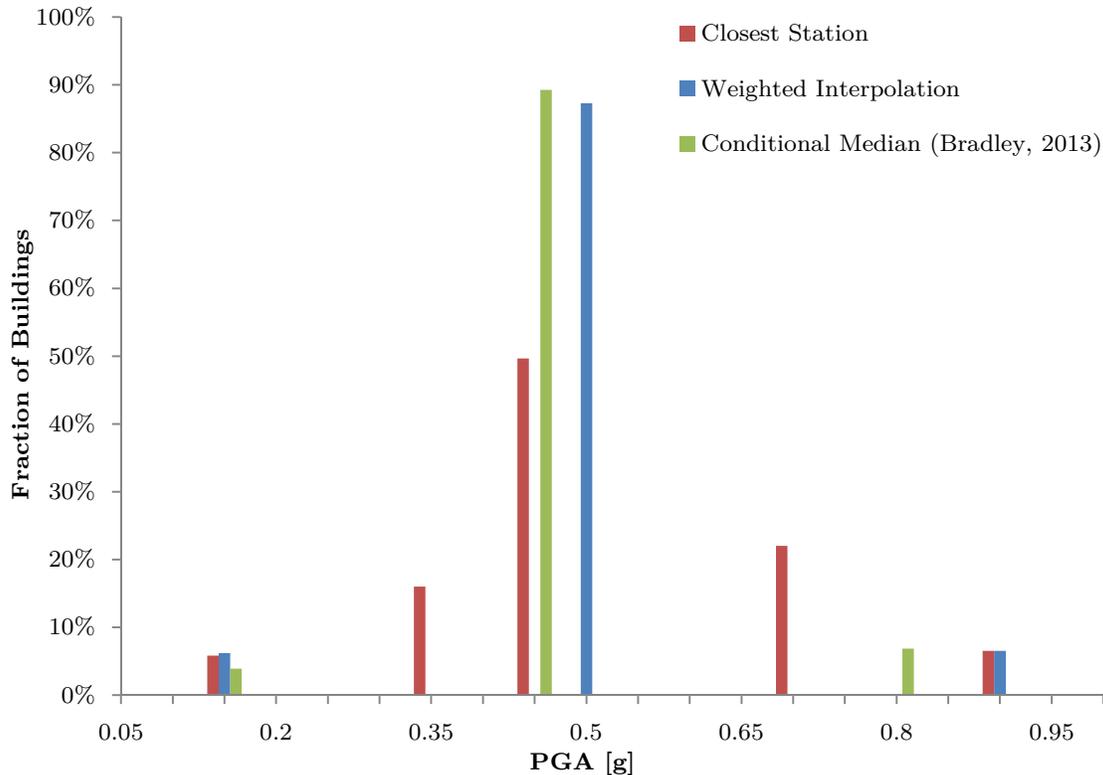


Figure 4.7 – Percentage of Buildings vs. PGA for Canterbury Database (Final)

The ‘Closest Station’ estimate clearly yields quite different results from the other two. However, the ‘Weighted Interpolation’ and ‘Conditional Median’ results are quite comparable, especially after consolidating the bins; in examining Figure 4.7, there appears to be about a 10% discrepancy between these two methods and some of this is due to the binning process itself. Another source of error is that the initial ground motion parameters may not come from the same source and could be subject to minor differences arising from processing of the accelerograms, which is especially true for PGA.

One noticeable difference between the Weighted Interpolation and the Conditional Median results is that there is much less variation in the latter (see Figure 4.6), which seems to contradict the earlier discussion on spatial variability. One possible explanation is the underlying assumption of a single representative soil condition for the generation of the PGA map.

In a forensic-type analysis of a specific building, only the most accurate of estimates would be appropriate. The data used herein, however, contains hundreds (if not thousands) of samples and we are primarily interested in the average damage/ground motions (as will be seen in Section 4.8). As such, it was concluded that the Weighted

Interpolation method provided an acceptably accurate estimate for the purposes of this study. The decision to use the Weighted Interpolation method (in lieu of the more rigorous Conditional Median method) represented a significant simplification, as it eliminated the need to generate conditional median maps in terms of $S_a(1)$ for all of the databases.

4.6 Developing DPMs & Fragility Curves from Damage Statistics

With the damage data collected and the ground motions assigned to each building, the next step is to compile the statistics in a manner that shows the relationship between ground motions and damage. The two most common forms are damage probability matrices (DPMs) and fragility curves. This section describes how the data was processed into the necessary forms and how the fragility curves were fit to the data.

4.6.1 Processing The Damage Data

This section discusses the process in terms of the ATC-13 damage states, but the methodology is valid regardless of the chosen damage scale. Refer to the previously shown Table 4.3 as an example DPM (note that the actual numbers of buildings are not shown – only the percentages). The DPM is constructed in the following manner:

- 1) The earthquake intensities are discretized into bins (eg. $0.3g < PGA < 0.4g$); *note: if using MMI, the “discretization” would simply be by MMI Level*
- 2) The damage measure is also discretized into bins. In this case, the discretization is already completed by the data collection method, in the form of the ATC-13 Damage States
- 3) The total number of buildings in each intensity/damage bin is calculated
- 4) For each intensity level, the number of buildings (#bldgs) in each damage state is then normalized by the total number of buildings in this intensity bin, giving a percentage of buildings (%bldgs) in each damage state for each intensity
- 5) Calculate the Mean Damage Factor (MDF) by multiplying the %bldgs in each damage state by its associated Central Damage Factor (CDF)
- 6) Finally, an optional step is to compute the sample standard deviation of the MDF at each intensity by taking the second moment of each damage state about the MDF (i.e. $S_{MDF} = \frac{\sum[(CDF_{DS=i} * \#Bldg_{SDS=I} - MDF)^2]}{\sum[\#Bldg_{SDS=I} - 1]}$)

This level of processing alone is sufficient to make several observations; for example, one could note that the MDF at a given intensity is much lower for a set of modern steel buildings than for a set of older URM buildings (i.e. the modern buildings suffered less damage, although the difference in replacement value also plays a role). Further processing is then performed to obtain the desired fragility curves.

The fragility curve for each damage state is obtained in the following manner and is illustrated in Figure 4.8, below. It is the cumulative probability distribution curve given in the lower right hand corner of the figure, and the process of translating the data is as described below:

- 1) For the given damage state, the fraction (or percent) of buildings equaling or exceeding the DS is calculated for each intensity level (in this case, PGA); the calculated values represent the conditional probability of being in this DS or greater, given the intensity level
- 2) The process is repeated for each damage state and results are plotted on a graph of PGA versus Probability of Reaching/Exceeding the given DS; note: the PGA to be plotted is the mean for the bin (eg. for the bin of $.3g < \text{PGA} < .4g$, the results are plotted at $.35g$)
- 3) The data is then fitted with an appropriate cumulative probability distribution. This may be accomplished in several ways and is discussed next

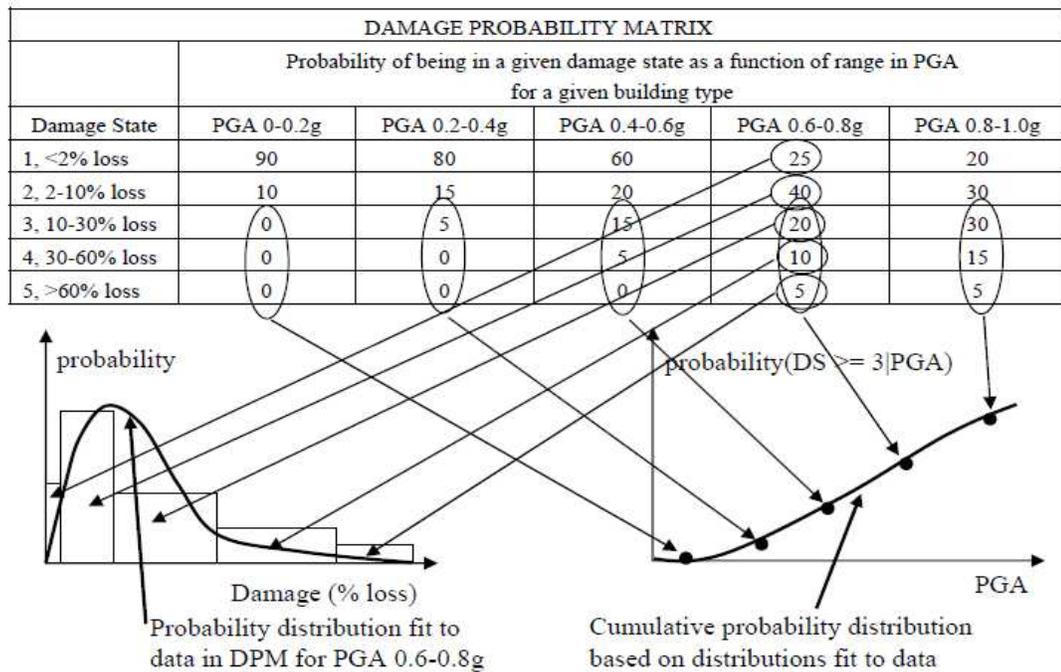


Figure 4.8 – Example Construction of Fragility Curve

(From King 2005)

4.6.2 Curve Fitting

With the data translated into the appropriate form, the next step is to fit the cumulative probability distribution. Various probability distributions have been used in the literature: in ATC-13, the beta distribution was used, while others have used the lognormal distribution (King, et al. 2005). It was noted that the primary reason for

using the beta distribution in ATC-13 was the fact that the lognormal distribution did not fit the data well at the lower damage states.

There are also many ways of fitting the chosen distribution to the data. Some common methods are as follows:

- **Moment Matching** – In this methodology, k number moments (i.e. mean, variance, skewness, kurtosis) of the distribution are matched to the sample data, where k is the number of unknown distribution parameters. For the lognormal distribution, there are two unknown parameters (μ, σ) , so typically the first two moments are matched. Other moments could be matched instead
- **Moment Optimization** – Exact matching of selected moments as noted above will not necessarily give appropriate matches for the remaining (unmatched) moments. This method aims to address this by instead minimizing the total mismatch of all the moments and is therefore an optimization method. A common objective function is $argmin\{(m-\mu)^2 + (s^2-\sigma^2)^2 + \dots\}$ (where $m, s \equiv$ sample estimates; $\mu, \sigma \equiv$ distribution parameters)
- **Nonlinear Regression** – In regression analysis, we seek to minimize the errors between the observed data and the predicted values of the dependent variable (i.e. the Y variable, or in this case the cumulative probability from the lower bound to X). The most common method is the “least squares” method, in which the objective is simply to minimize the sum of the errors (i.e. the difference between the data and the fitted curve) squared
- **Maximum Likelihood Estimation (MLE)** – In MLE, we seek to obtain the parameter values, which make observed data the most probable (i.e. the likelihood function is maximized). This method is not reviewed here except to note that, for the beta distribution, the method is quite sensitive to the upper and lower bounds and thus is not recommended for curve-fitting of the beta distribution (Simaan and Halpin 1994)

For this study it was decided to do fitting by nonlinear regression. One of the major advantages of using nonlinear regression is that the individual terms in the objective function (i.e. the sum of errors to be minimized) can be weighted. As the number of buildings within a bin could vary from less than fifty to over a thousand, it was decided to weight the fitting by the number of buildings in the bin. The weighting factor was defined as follows:

$$WF_i = MDF_i / SE_i \tag{4-4}$$

Where:

WF \equiv Weighting Factor for bin ‘i’

MDF_i \equiv Mean Damage Factor for the bin ‘i’

SE_i \equiv Standard Error of the Mean for bin ‘i’

Note that the standard error of the mean is defined as the sample standard deviation divided by the square root of the number of measurements in the sample. The theory is discussed in textbooks on elementary statistics (Navidi 2010), but the essence is that for larger samples of the same population, the variation in the mean from sample to sample is quite small even if the distribution of the population is quite wide.

The MDF is included in the weighting as a method of normalizing the variation, similar to a coefficient of variation. It should be noted that, in some instances, the weighting was manually adjusted where the data appeared unreliable or where there were very few buildings, all with mostly the same damage rating (which would give a SE near zero and therefore a large weighting factor). In these cases, the weighting was reduced to zero, or some nominally small value so as not to affect the results – such data points will be highlighted in the subsequent results, presented in Section 4.8.

As aforementioned, both the beta distribution and the lognormal distribution were candidates for the fitting. Background on the distributions is commonly available in textbooks on statistics (Navidi 2010), but the major difference is that the lognormal distribution is defined by two parameters (for example the mean and standard deviation), while the beta distribution is defined by four parameters, which offers more flexibility in the shape of the distribution. The beta distribution is also defined over a finite range: for example, one could prescribe the PGA at which 0% and 100% of the structures reach a certain damage state. In this study, it was decided to prescribe a minimum of $S_a(1)=0g$ and to leave the maximum value unconstrained.

Both the lognormal and beta distributions were fitted and the results were compared. Figure 4.9 provides an example of the resulting curves for the MDF versus $S_a(1)$ from the Canterbury buildings (see Section 4.8.4). For each data set, it was found that there was little difference between the two distributions in the region of the data, which is intuitive. In the regions beyond the data, the lognormal distribution consistently fell below the beta distribution, presumably because the shape of the beta distribution was less constrained. As the higher values of the beta distribution were more consistent with physical insight (i.e. at an $S_a(1)$ of 5.0g one would expect all unretrofitted URM buildings to be destroyed) it was decided to use this distribution in the fitting. For

practical purposes, however, the choice between the Beta and Lognormal distribution is of minor consequence because the probability of occurrence of such extreme shaking is low enough so as to not affect the resulting loss estimates.

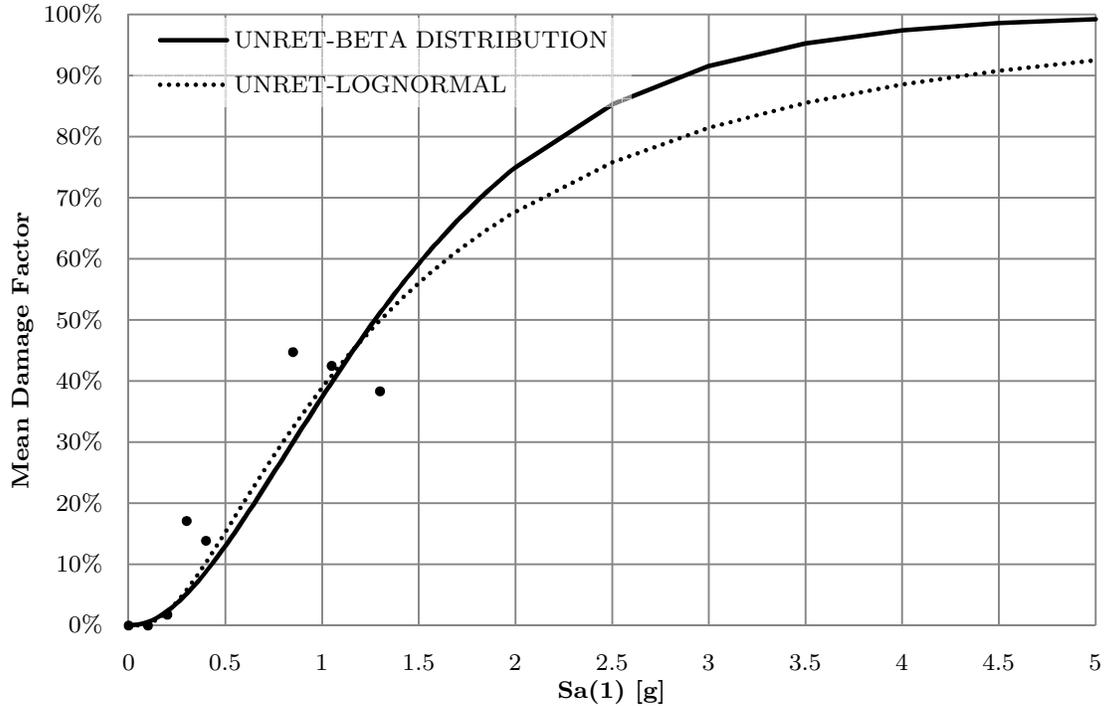


Figure 4.9 – Beta vs. Lognormal Distribution for MDF of Canterbury Buildings

4.7 Literature Review and Summary of Available Data

A significant body of work on URM damage statistics was completed during the 1980's and 1990's as a result of the major earthquakes in California at the time. More recently, a significant amount of data on URM buildings was collected from the 2010/2011 Canterbury earthquakes. Because there is no similar data on western Canadian URM buildings, these sources are thought to be the best available data. The available data and results were in various forms and, as such, a significant amount of effort was devoted to reviewing these results and working with the raw data to produce additional results that were of interest to this study. This section reviews the work that has been completed by others. The reviews are kept reasonably brief and the reader may refer to the original documents (Deppe 1988, Lizundia, Dong and Holmes 1993, Wiggins, Breall and Reitherman 1994, Rutherford & Chekene 1997, Ingham and Griffith 2011a, 2011b) for more information. Section 4.8 presents the new results that were prepared by the author. Both sections are organized by the subject earthquakes.

4.7.1 Whittier 1987 Earthquake

The Whittier Narrows earthquake occurred on October 1, 1987. It was a $M_L=5.9$ event and the epicenter was about 15km east of downtown Los Angeles. Figure 4.10 shows a map of instrumental intensity, produced by the United States Geological Survey.

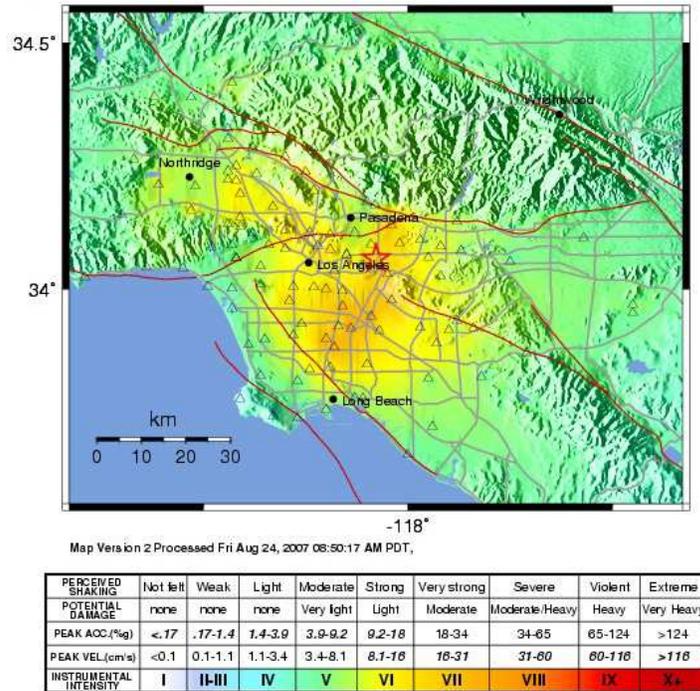


Figure 4.10 – Instrumental Intensity Map – Whittier Narrows Earthquake

Source: http://earthquake.usgs.gov/earthquakes/shakemap/sc/shake/Whittier_Narrows

4.7.1.1 Summary of Damage Survey

Two URM damage surveys were performed: one by Deppe (1988) and another by Wiggins et al. (1994). The survey by Deppe covered 2431 of the 7300 URM buildings in Los Angeles, but the damage assessment was limited to:

- 1) Undamaged
- 2) Damaged, including nonstructural, but entirely functioning (not vacated)
- 3) Vacated (wholly or partially)

Deppe estimated that approximately 1100 buildings had been fully strengthened. Rutherford and Chekene (1997) later estimated this number to be only approximately 800. Buildings were sorted into those that were strengthened (to the city’s mandatory retrofit standard, Division 88, 1985) and those that were unstrengthened or partially strengthened (eg. tension ties only). More detailed information is provided by Deppe (1988), but the following key observations summarize the findings:

- Unstrengthened/partially strengthened buildings were 50% more likely to have observable damage; this includes any damage ranging from non-structural cracking in, say, plaster ceilings, to partial collapse
- Unstrengthened buildings were 170% more likely to have been wholly or partially vacated (presumably due to damage)
- Residential buildings were approximately 150% more likely to have been partially or wholly vacated

The second database, by Wiggins (1994), was more detailed. It was based on ATC-13 damage levels and MMI was the intensity measurement. However, the intensity assigned to each building was based on Wiggins' own attenuation equation rather than measured values as shown in the map above. Wiggins accounted for soils in his assessment, but only on a regional basis by assigning soil types using zip codes and his own map of regional soil types. Table 4.5, Table 4.6, and Table 4.7, show the resulting breakdown by MMI and ATC-13 damage state for unstrengthened, partially strengthened, and fully strengthened buildings (to Los Angeles requirements), respectively. As can be seen, the vast majority (over 95%) of the buildings fell into the MMI VII range. However, the data set is valuable because it has a reasonable number of buildings in the partially strengthened state, which was found to be rare in the literature that was reviewed. This data will be further processed in Section 4.8.1.

Table 4.5 – Whittier DPM – Unstrengthened

(From Wiggins, 1994)

Damage State		Geom. DF	Number of Buildings in MMI					
			5.5-6.0	6.0-6.5	6.5-7.0	7.0-7.5	7.5-8.0	8.0-8.5
1	None (0%)	0%	9	29	18	213	800	0
2	Slight (0-1%)	0.5%	1	1	0	45	66	0
3	Light (1-10%)	3.2%	0	3	1	52	230	0
4	Moderate (10-30%)	17.3%	0	0	2	29	122	0
5	Heavy (30-60%)	42.4%	0	0	1	2	26	0
6	Major (60-100%)	77.5%	0	0	1	2	6	0
7	Destroyed (100%)	100%	0	0	0	0	0	0
MDF=			.06%	.031%	6.86%	2.71%	3.56%	--

Table 4.6 – Whittier DPM – Partially Strengthened

(From Wiggins, 1994)

Damage State		Geom. DF	Number of Buildings in MMI					
			5.5-6.0	6.0-6.5	6.5-7.0	7.0-7.5	7.5-8.0	8.0-8.5
1	None (0%)	0%	1	1	4	39	132	1
2	Slight (0-1%)	0.5%	0	0	0	6	21	0
3	Light (1-10%)	3.2%	2	1	1	9	57	0
4	Moderate (10-30%)	17.3%	0	0	0	5	29	0
5	Heavy (30-60%)	42.4%	0	0	1	0	2	0
6	Major (60-100%)	77.5%	0	0	0	0	0	0
7	Destroyed (100%)	100%	0	0	0	0	0	0
MDF=			2.11%	1.59%	7.69%	2.01	3.23%	0.00%

Table 4.7 – Whittier DPM – Fully Strengthened

(From Wiggins, 1994)

Damage State		Geom. DF	Number of Buildings in MMI					
			5.5-6.0	6.0-6.5	6.5-7.0	7.0-7.5	7.5-8.0	8.0-8.5
1	None (0%)	0%	2	6	9	98	256	0
2	Slight (0-1%)	0.5%	0	0	0	6	11	1
3	Light (1-10%)	3.2%	0	0	1	11	53	0
4	Moderate (10-30%)	17.3%	0	0	0	0	16	0
5	Heavy (30-60%)	42.4%	0	0	0	2	4	0
6	Major (60-100%)	77.5%	0	0	0	0	0	0
7	Destroyed (100%)	100%	0	0	0	0	0	0
MDF=			0.00%	0.00%	0.33%	1.06%	1.83%	0.5%

4.7.2 Loma Prieta 1989 Earthquake

The Loma Prieta earthquake occurred on October 17, 1989. It was a $M_w=6.9$ event and the epicenter was about 14km northeast of Santa Cruz. Figure 4.11 shows a map of instrumental intensity, produced by the United States Geological Survey.

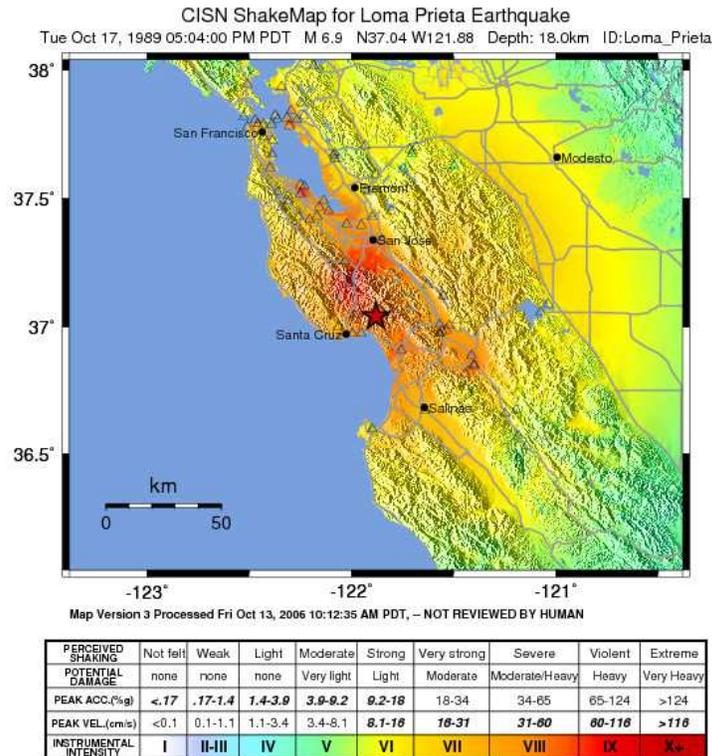


Figure 4.11 – Instrumental Intensity Map – Loma Prieta Earthquake

Source: <http://earthquake.usgs.gov/earthquakes/shakemap/nc/shake/LomaPrieta/>

4.7.2.1 Summary of Damage Survey

Lizundia (1991) compiled damage statistics from various cities throughout California, stretching approximately from Salinas (in the south) to Oakland (in the north). The survey essentially covered all areas of MMI VII and greater. Lizundia et al. (1993) later conducted various statistical analyses on the damage statistics. The different levels of data are noted as follows:

- 1) Level 1 data includes the most buildings (4824 throughout 113 cities); however, it is limited to a “general damage status,” which is the number of buildings that were damaged, vacated, or demolished
- 2) Level 2 data includes 2356 buildings throughout nine cities; it includes ATC-13 damage states, basic building information such as the number of storeys and occupancy, and soils information

- 3) Level 3 data covers only buildings in San Francisco, including 896 buildings; it includes more detailed information on cracking patterns, veneer failures, corner damage, and more

The vast majority of the buildings in this survey were unstrengthened, as most communities had not yet implemented mandatory retrofit ordinances. Strengthening status statistics from the Level 2 database are as follows:

- 2110 buildings are noted as unstrengthened
- 32 buildings are noted as partially strengthened
- 29 buildings are noted as fully strengthened
- 1310 buildings have unknown strengthening statuses

The analyses undertaken by Lizundia et al. (1993) focused primarily on unstrengthened buildings and, therefore, excluded buildings in the latter 3 categories. Note that an additional 1085 buildings from San Francisco were excluded by Lizundia et al (1993) because these buildings had received parapet strengthening. For this study, the risk reduction afforded by bracing parapets was of great interest, as the industry sponsor (VCHT, as introduced in Chapter 1) was in the process of developing a new incentive program for parapet bracing at the time of this study. Analysis of this data, and more, was completed by the author and is presented in Section 4.8.

4.7.2.2 Level 1 Data

The level 1 data is not directly meaningful in terms of quantifying damage, as it does not provide numeric estimates of loss/damage. However, it is useful because it provides results that are easily appreciated in terms of their impact on a community. Figure 4.12 is reproduced directly from Lizundia et al. (1993) and shows the fraction of buildings that were be damaged, vacated, and demolished. Although not noted in the original document, it appears that the figures did not include data from San Francisco or Oakland. This is presumably because of the large number of buildings (together, they represent approximately 44% of the 4800+ building) and because the soils were known to be quite variable. The key point of the figure is that there is a large jump in damage at MMI VIII. It should be noted that the MMI VIII sample represents just 86 buildings in total, while the MMI VII and VI samples represent 1108 and 1499, respectively. Nonetheless, one can imagine the impact on the communities that experienced MMI VIII, such as Santa Cruz, Los Gatos, or Watsonville.

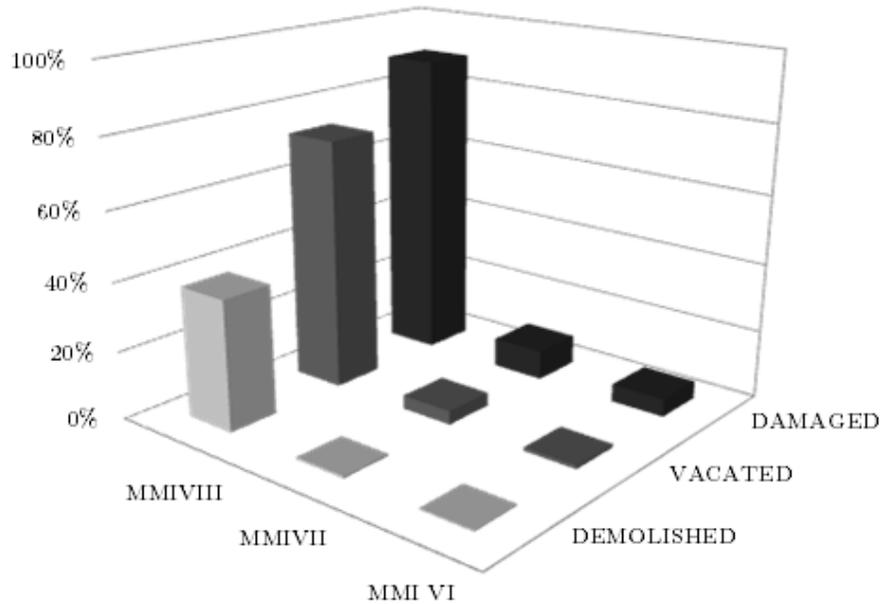


Figure 4.12 – Level 1 Damage Data for Loma Prieta Earthquake
(Reproduced from Lizundia et al., 1993)

4.7.2.3 Level 2 Data (ATC-13 Damage Statistics)

The level 2 data comprised much of Lizundia’s work and resulted in damage probability matrices, based on ATC-13 damage states and MMI. Unlike the Whittier data (see Section 4.7.1), the Level 2 Loma Prieta data was binned into whole-MMI ranges, rather than half-MMI ranges (such as 6.0-6.5). Although the earthquake produced a relatively high level of shaking intensity near the epicenter, much of the data again fell into the lower MMI ranges, with 91% of the data in the VI-VII range; only 8% (179 buildings) fell into the VIII range, and 1% (27 buildings) fell into the IX range. The resulting DPM from Lizundia et al. (1993) is reproduced here as Table 4.8, except that the MDF is recalculated using the geometric DF rather than the CDF.

Table 4.8 – Loma Prieta DPM - Unstrengthened
(Modified From Lizundia et al., 1993)

Damage State		Geom. DF	Number of Buildings in MMI			
			VI	VII	VIII	IX
1	None (0%)	0%	409	1199	82	0
2	Slight (0-1%)	0.5%	98	141	20	6
3	Light (1-10%)	3.2%	53	1111	29	14
4	Moderate (10-30%)	17.3%	24	34	12	3
5	Heavy (30-60%)	42.4%	5	40	33	3
6	Major (60-100%)	77.5%	3	8	2	1
7	Destroyed (100%)	100%	0	0	1	0
MDF=			1.82%	2.17%	10.98%	11.27%

Although this data set is also somewhat limited to lower levels of shaking intensity, it is still quite a bit more diverse in ground motion intensity than was the data from the Whittier earthquake. Since 95% of the Whittier data fell into the MMI VII range (7.0-7.5 and 7.5-8.0), we can make a conclusive comparison only for this range: combining the above noted bins from Table 4.5 and recalculating results in a MDF equal to 3.18%. This compares reasonably well to the corresponding value for Loma Prieta of 2.17%, considering the many uncertainties involved (eg. data collection, soils, building construction, interpolation of ground motion IM).

With regards to the MMI VI data, one might speculate that the Whittier buildings performed markedly better at MMI 6.0-6.5 and markedly worse at 6.5-7.0. However, if we again combine these two bins the resulting MDF would be 3.0%, which is much closer to the Loma Prieta value of 1.82%. This suggests that perhaps Wiggins (1994) binned his data too finely – especially considering the following:

- MMI ranges are much finer at the lower end of the scale (eg. VI \equiv PGA .09-.18g, IX \equiv PGA .65-1.24g)
- Wiggins' intensity measure was based on calculated rather than measured values
- Soil conditions were assigned based only on regional mapping

4.7.2.4 Damageability Effects of Specific Building Characteristics

One of the key limitations of a fragility-based approach such as ATC-13 is that all structures in the sample are assumed to have the same characteristics. In an effort to investigate the effects of various building characteristics/irregularities, Lizundia et al. (1993) conducted statistical testing (t-testing of the mean damage factors) for various subsets of the sample to determine whether or not certain characteristics had a significant effect on the performance of the buildings. This portion of the study considered only San Francisco buildings that had not received parapet strengthening. The following was investigated:

- Site soils
- Storey height
- Occupancy
- Building configuration (eg. square, rectangular, irregular)
- Diaphragm ratio (the plan aspect ratio of length to depth)
- The presence of a “soft storey”
- Year of construction
- Number of storeys

Tables 4.9 to 4.11 show the resulting MDF for the various subsets and the number of buildings in the sample. Soil type was by far the most significant attribute; in fact, its effect was so pronounced that it was necessary to sort the buildings by soils, before analyzing other attributes (Lizundia, et al. 1991). Storey height was also significant. This seems reasonable, since both aspects are key issues in retrofit design. Occupancy was found to be a significant attribute; this is presumably due to the different structural forms that are associated with these occupancies. This again appears consistent with engineering judgement, as Rutherford & Chekene (1990) separated buildings into occupancy-related prototypes for loss estimates. Lastly, the number of stories was a significant attribute: one to three storey buildings had a lower MDF than four storey buildings. However, the trend was inconsistent in that 5, 6, and 7 storey buildings appeared to suffer less damage (see Table 4.12).

While statistically significant differences were found for some of the remaining attributes, the damage trends were not consistent enough to warrant further investigation as part of this study and, as such, they are not shown.

Table 4.9 – Soil Type vs. MDF

(From Lizundia, 1993)

Soil Type	MDF	#Bldgs
1 (Rock/Stiff Soil <200 ft)	0.49%	45
2 (Stiff Soil >200 ft)	4.78%	660
3,4 (medium/soft clay)	6.68%	171

Table 4.10 – Wall Height vs. MDF

(From Lizundia, 1993)

Wall Height	MDF	#Bldgs
< 16 feet	3.95%	637
> 16 feet	7.90%	239

Table 4.11 – Occupancy vs. MDF

(From Lizundia, 1993)

Occupancy	MDF	#Bldgs
Residential, Assembly	4.42%	301
Commercial, Office	2.69%	271
Industrial	6.63%	304

Table 4.12 – Storeys vs. MDF

(From Lizundia, 1993)

Number of Storeys	MDF	#Bldgs
1, 2, 3	4.60%	695
4	7.86%	95
5, 6, 7	2.04%	83
8	11.2%	3

While these statistics have provided some insight into the attributes that may affect the vulnerability of a given building, one key shortcoming is that no results are provided for buildings with more than one of the aforementioned attributes – what about a four-storey building of commercial occupancy, with storey heights less than 16 feet versus a 2-storey building of industrial occupancy, with storey heights greater than 16 feet? It is unlikely that the answer is a direct superposition of the results, and the attributes themselves may be correlated.

4.7.3 Northridge 1994 Earthquake

The Northridge earthquake occurred on January 17, 1994. It was a $M_w=6.7$ event and the epicenter was about 32km northwest of Los Angeles. Figure 4.13 shows a map of instrumental intensity, produced by the United States Geological Survey.

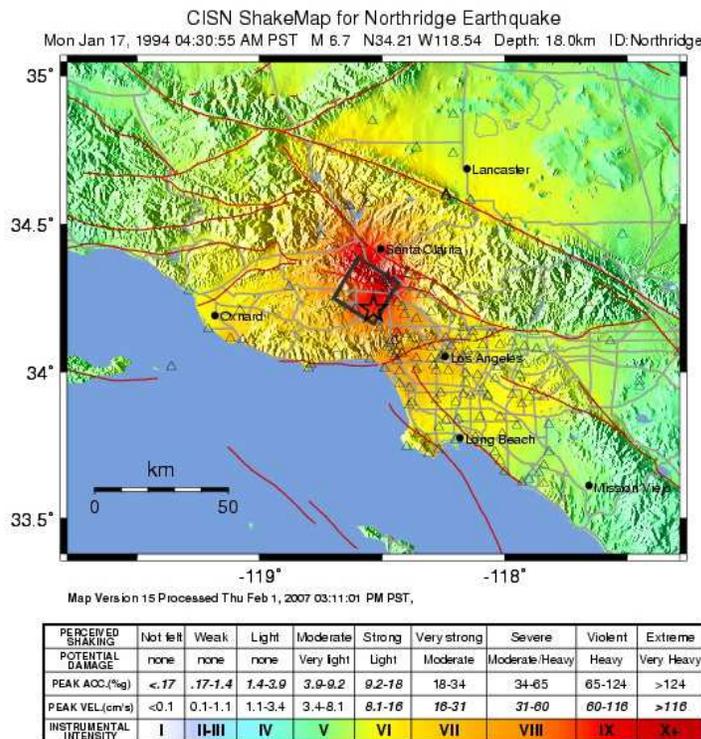


Figure 4.13 – Instrumental Intensity Map – Northridge Earthquake

Source: <http://earthquake.usgs.gov/earthquakes/shakemap/sc/shake/Northridge/>

4.7.3.1 Summary of Damage Survey

Rutherford and Chekene (1997) compiled damage statistics for Los Angeles, which were collected by the Los Angeles Building and Safety Department. The data was based on post-earthquake safety evaluations similar to ATC-20, including ATC-13 damage states and various damage descriptions. Unlike the Whittier earthquake, the majority of the URM buildings in Los Angeles had been seismically upgraded by this time, in accordance with the City’s mandatory retrofit ordinance, Division 88. Unfortunately, only a modest sample of the total population was assessed and the sample was biased in that it focused on the most heavily damaged areas of the city (Rutherford & Chekene 1997). Data was collected as noted below:

- 751 of 5682 retrofitted buildings
- 93 of 703 unretrofitted buildings
- 8 of 61 partially retrofitted (tension tie only) buildings

Similar to many of the other damage surveys, the level of ground shaking experienced by the buildings was predominantly at the MMI VII level – Figure 4.14 shows the number of retrofitted buildings in each MMI category.

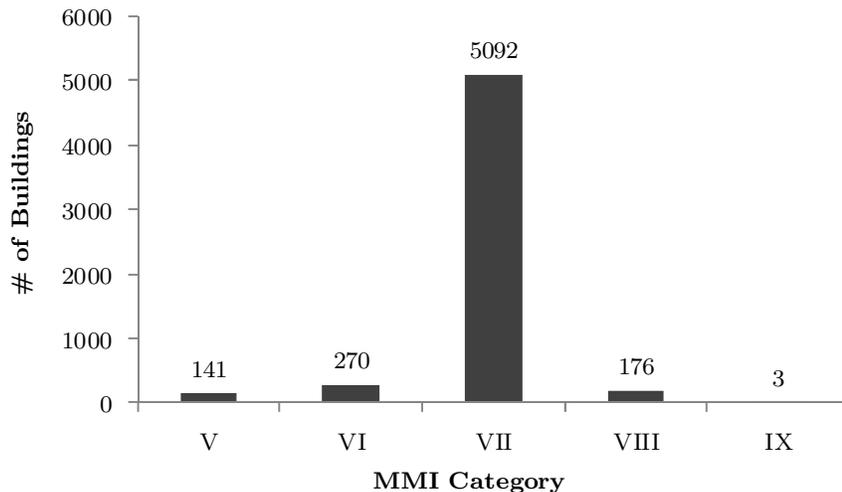


Figure 4.14 – MMI Breakdown for Northridge Earthquake
(Reproduced from Rutherford & Chekene, 1997)

As aforementioned, only a portion of these buildings were actually assessed. To address this issue, Rutherford and Chekene produced damage statistics for three scenarios:

- 1) For the inspected buildings only
- 2) For the entire population, assuming those not inspected had no damage
- 3) For the entire population, assuming some level of damage (dependent upon the ground motion intensity)

The third case fell closer to the ‘no damage’ case and Rutherford and Chekene (1997) noted that this was their best estimate of the damage.

4.7.3.2 ATC-13 Damage Statistics

ATC-13 damage statistics were generated, similar to those from the Loma Prieta and Whittier earthquakes. Table 4.13 provides the DPM for the third case (i.e. some damage assumed). Note that the geometric mean damage factor was again substituted for the central damage factor, as used in the original study. Also, the results are not plotted at MMI IX, since only three buildings were in the sample.

Table 4.13 – Northridge DPM - Strengthened

(Modified From Rutherford & Chekene, 1997)

Damage State		GMDF	Number of Buildings in MMI			
			V	VI	VII	VIII
1	None (0%)	0%	138	131	2339	58
2	Slight (0-1%)	0.5%	2	133	1503	58
3	Light (1-10%)	3.2%	1	5	1144	44
4	Moderate (10-30%)	17.3%	0	1	70	12
5	Heavy (30-60%)	42.4%	0	0	25	3
6	Major (60-100%)	77.5%	0	0	8	1
7	Destroyed (100%)	100%	0	0	3	0
MDF=			0.04%	0.37%	1.49%	3.30%

The original study also provided DPMs for unretrofitted and tension tie-only buildings. However, since the number of buildings actually inspected was so small, a ‘best estimate’ level of damage was not provided; only the case assuming no damage to inspected buildings was calculated. Because data from several other, more reliable, sources was reviewed, there was little motivation to make use of the Northridge data for unretrofitted or tension tie-only buildings and no further discussion is presented.

4.7.3.3 Damageability Effects of Specific Building Characteristics

Similar to the work of Lizundia (1993), the effects of specific building characteristics were investigated by analyzing the appropriate subsets of the sample. The following characteristics were investigated:

- 1) Number of storeys
- 2) Presence of a basement
- 3) Horizontal Aspect Ratio (ratio of plan dimensions of the building footprint)
- 4) Vertical Aspect Ratio (ratio of height to least dimension in elevation)

Unfortunately information on soils was not available. However, the subject buildings are concentrated in an area of relatively uniform soil conditions. Kalkan et al. (2010) notes that extensive areas of the Los Angeles basin are underlain by Pleistocene alluvium (mean $V_{s30}=377\text{m/s}$), while the center is underlain by younger alluvium (mean $V_{s30}=287$). This indicates that most of the buildings were likely founded on Site Class D, or perhaps Site Class C, soils. In addition, the Los Angeles area is well-instrumented (ATC 2001), and so the effects of soils were deemed to be suitably well-captured for characterizing average damage. Tables 4.14 to 4.17 below show the mean damage factors for the various subsets. Unlike the Loma Prieta set, a breakdown by ground motion intensity was included. Note that for items 1) and 4), uninspected buildings were assumed to have no damage. For items 2) and 3), only inspected buildings were included. The horizontal and vertical aspect ratios were analyzed in terms of both $S_a(0.3s)$ and $S_a(1.0s)$; the results for $S_a(1.0s)$ are presented here, because this is consistent with our selected ground motion intensity measure.

Table 4.14 – Number of Storeys vs. MDF

(Reproduced From Rutherford & Chekene, 1997)

Number of Storeys	$S_a(0.3s)$ [g]						Total	
	0.35-0.50		0.50-0.65		0.65-1.225		MDF	#Bldgs
	MDF	#Bldgs	MDF	#Bldgs	MDF	#Bldgs		
One to Three	0.77%	3968	0.98%	1055	2.34%	201	0.86%	5224
Four to Six	2.78%	342	1.01%	112	--	0	2.35%	454

Table 4.15 – Presence of Basement vs. MDF

(From Rutherford & Chekene, 1997)

Presence of Basement	$S_a(0.3s)$ [g]						Total	
	0.35-0.50		0.50-0.65		0.65-1.225		MDF	#Bldgs
	MDF	#Bldgs	MDF	#Bldgs	MDF	#Bldgs		
With	6.68%	191	2.39%	60	12.2%	5	5.78%	256
Without	8.98%	170	9.64%	74	7.71%	34	9.00%	278

Table 4.16 – Horizontal Aspect Ratio vs. MDF

(From Rutherford & Chekene, 1997)

Horizontal Aspect Ratio	$S_a(1.0s)$ [g]						Total	
	0.075-0.20		0.20-0.35		0.35-0.75		MDF	#Bldgs
	MDF	#Bldgs	MDF	#Bldgs	MDF	#Bldgs		
< 2.0	5.72%	165	8.99%	134	8.57%	22	7.28%	321
> 2.0	5.82%	186	8.44%	185	12.7%	34	7.51%	405

Table 4.17 – Vertical Aspect Ratio vs. MDF

(From Rutherford & Chekene, 1997)

Vertical Aspect Ratio	$S_a(1.0s)$ [g]						Total	
	0.075-0.20		0.20-0.35		0.35-0.75		MDF	#Bldgs
	MDF	#Bldgs	MDF	#Bldgs	MDF	#Bldgs		
< 0.5	0.45%	3036	1.27%	1330	3.25%	168	0.80%	4534
> 0.5	0.94%	732	3.13%	395	4.06%	17	1.75%	1144

Of the four items, only two appear particularly conclusive: 2) the presence of a basement and 4) the vertical aspect ratio. The former of these two is interesting, but not particularly useful. It is postulated that this result is due to some correlation of other characteristics with the presence of a basement, such as quality of construction.

With regards to the number of storeys, the MDF for the two subsets is markedly different at just slightly different ground motion intensities, which seems unlikely. While it seems intuitive that number of storeys would be a significant characteristic, one must acknowledge that two buildings with the same number of storeys could actually be quite different in form – one building could be just 20-30 feet wide, while another could run an entire city block. By contrast, the vertical aspect ratio appears to be a much better indicator of vulnerability.

With regards to the horizontal aspect ratio, one may have expected to see a significant impact here as well. However, there is virtually no difference except at the highest level of shaking, which is comprised of a very small sample. This could easily have been skewed by one or two buildings with high ATC-13 damage state classifications. Based on the results, it appears that horizontal aspect ratio is not a significant predictor of vulnerability. One may rationalize this by noting that current retrofit provisions do attempt to address excessive diaphragm deflections.

4.7.3.4 Correlation between Ground Motion and Specific Damage Types

One final item investigated by Rutherford & Chekene (1997) was concerned with determining the ground motion intensities at which specific types of damage occur. To this end, Rutherford & Chekene provide the PGA, $S_a(0.3s)$, and $S_a(1.0s)$ at which one percent of the total inventory is affected and at which a sharp jump in the frequency of damage occurs. Items investigated included:

- Building/Storey Leaning
- Foundation Damaged
- Roof/Floors Damaged

- Columns/Pilasters/Corbels Damaged
- Diaphragms/Horizontal Bracing Damaged
- Walls/Vertical Bracing Damaged
- Cladding/Glazing Damaged
- Shear Cracks
- Corner Damage

Little difference was observed in the points at which the various types of damage occurred. In general, 1% of the inventory was damaged at a $PGA=.15-.20g/S_a(1.0)=.15-.20g$ and a “sharp jump” occurred at a $PGA=.35-.40g/S_a(1.0)=.40-.45g$. Rutherford and Chekene (1997) noted that the jump occurred at a lower ground motion intensity for wall cracking, but that this may have been due to the fact that wall cracking is so easily observed.

4.7.4 Canterbury 2010/2011 Earthquake Swarm

The Canterbury earthquake swarm consisted of several earthquakes, over a period of more than a year. 41 earthquakes of magnitude 5 or greater were detected from September 2010 through January 2011 (Nicholls 2012). The four largest events were as follows (New Zealand local date):

- $M_w7.1$ on September 4, 2010, 40km west of Christchurch
- $M_w6.3$ on February 22, 2011, 10km southeast of Christchurch
- $M_w6.4$ on June 13, 2011, 10km southeast of Christchurch
- $M_w5.8$ on December 23, 2011, 26km east of Christchurch

The February earthquake was by far the most damaging, although significant damage was also sustained by URM buildings in the September and June events (Moon, et al. 2014). Because damage was so heavy in the February event, only the September and February events are of particular interest to this study. Figures 4.15 and 4.16 show USGS-generated maps of instrumental intensity for the September event (also known as the Darfield earthquake) and the February event (also known as the Christchurch earthquake), respectively.

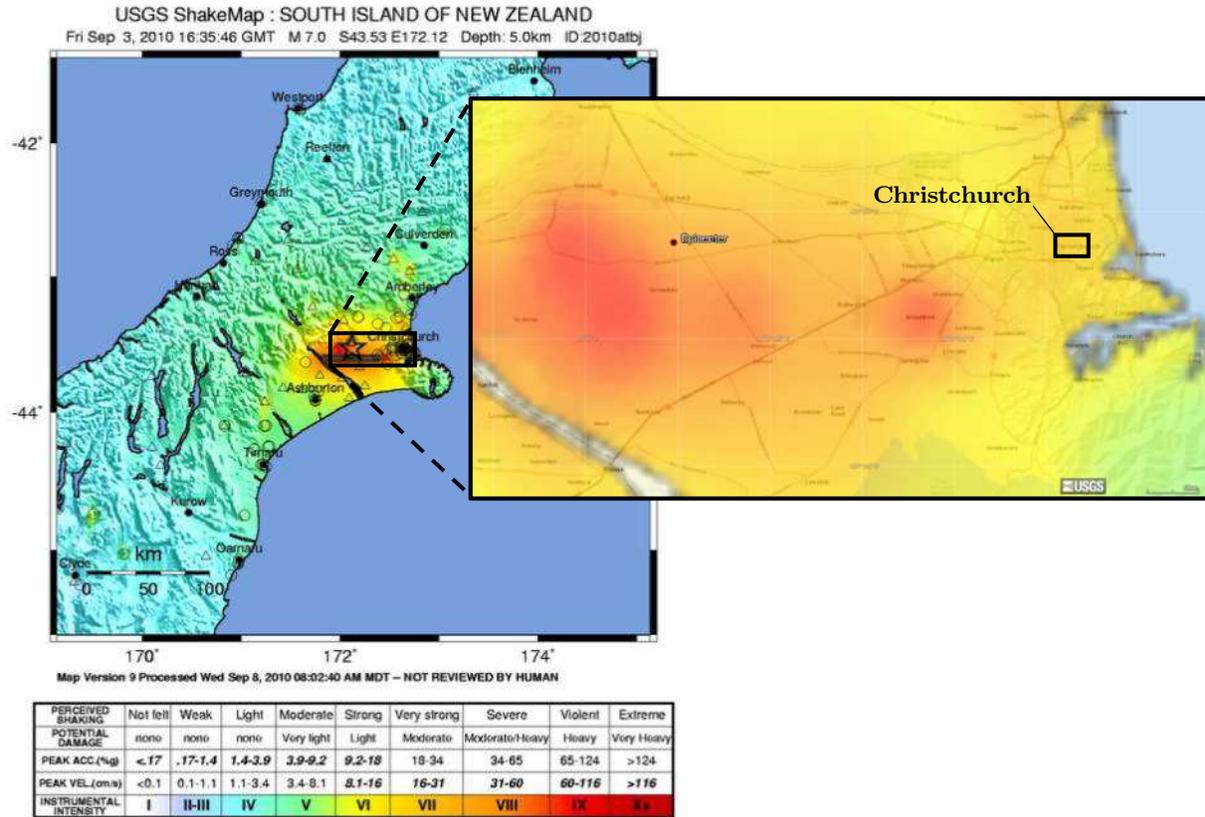


Figure 4.15 – Instrumental Intensity Map – September (Darfield) Earthquake

Modified From: <http://earthquake.usgs.gov/earthquakes/shakemap/global/shake/2010atbj/>

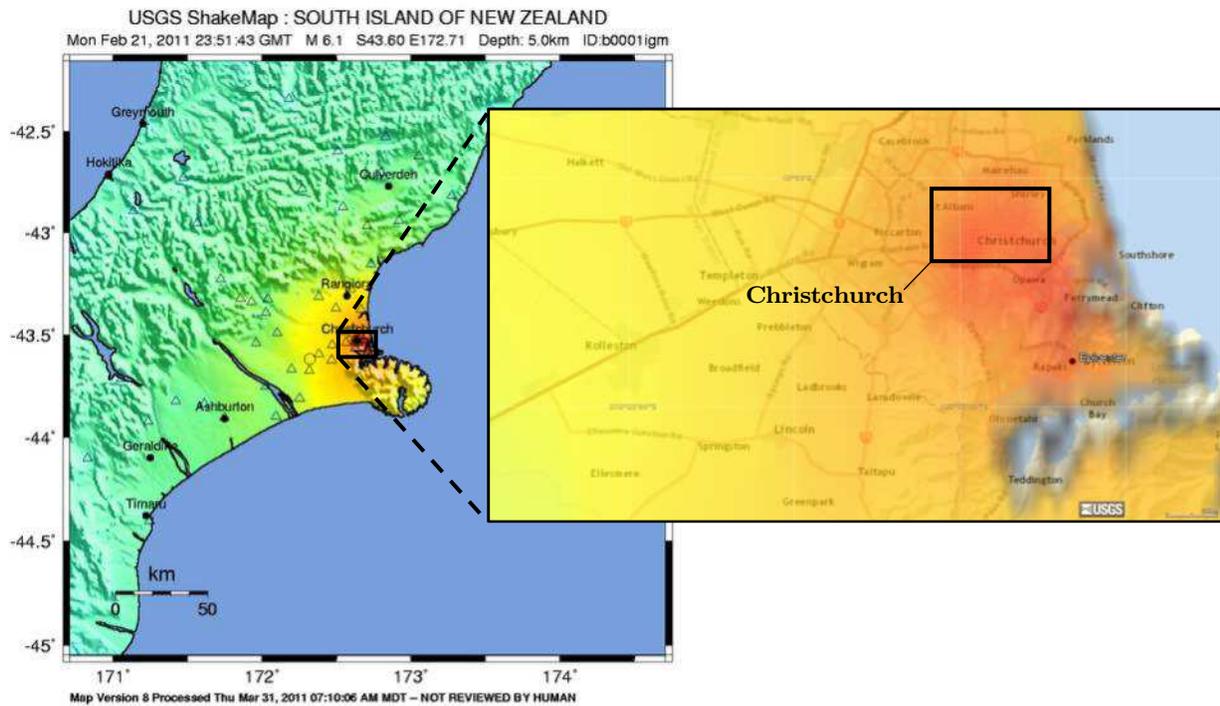


Figure 4.16 – Instrumental Intensity Map – February (Christchurch) Earthquake

Modified From: <http://earthquake.usgs.gov/earthquakes/shakemap/global/shake/b0001igm/>

In Christchurch, where the majority of the buildings were located, shaking was predominantly at the MMI VII level for the September earthquake. For the February earthquake, shaking was predominantly at the MMI VIII/IX level. As will be seen, the fact that many of the buildings were subjected to both earthquakes presents unique opportunities and challenges in compiling damage statistics. Another important point is that this was the only damage survey to contain a significant number of buildings in the MMI IX range.

4.7.4.1 Summary of Damage Survey

A damage survey of URM buildings was completed by researchers at the Universities of Auckland, in New Zealand, and Adelaide, in Australia (Ingham and Griffith 2011a, 2011b). The first report characterizes and generally assesses the observed performance of URM buildings, particularly in the September earthquake. The second report focuses solely on the performance of buildings in the February earthquake, with particular attention to the effects of strengthening. Most of the useful damage statistics are provided in the second report. Note that the information is not necessarily summarized here in terms of the separate reports. Rather, the body of work is reviewed as a whole and is presented in a way that lends most usefully to the study at hand.

The database consists of 626 URM buildings. The majority were clay brick buildings, with flexible timber diaphragms, but some stone masonry buildings were also present. According to Ingham and Griffith (2011a), there are an estimated 852 URM buildings in the Canterbury province (which more than encompasses the area affected by the earthquake), so the 626+ buildings is certainly a reasonable sample of the population. In terms of the Christchurch central business district, Ingham and Griffith (2011b) note that 370 out of 380 total URM buildings were reviewed following the February earthquake.

For the February event, the survey was quite detailed and involved descriptions of any retrofitting work, component specific damage descriptions, building usability placards based on NZSEE (2009) (similar to ATC-20), ATC-13 damage state classifications, and Wailes and Horner damage classifications. Data was acquired by (primarily) exterior reviews, observing buildings during demolition, aerial photograph, Google street view, and review of Christchurch City Council property records. For the September event only about 50% of the buildings were assessed for ATC-13 damage states, although placard data was available for about 80% of the buildings.

A variety of strengthening statuses were present, ranging from completely unstrengthened to strengthening equal or greater than current code for New Zealand. Ingham & Griffith (2011b) classified strengthening into three categories:

- 1) Parapet strengthening: including the following
 - a. the addition of a concrete ring beam, or
 - b. the addition of structural steel bracing back to the roof structure
- 2) Type A Strengthening: including the following
 - a. Installing connections between walls and the roof and floor systems so that walls no longer respond as vertical cantilevers
 - b. Stiffening of the roof and/or floor diaphragms
- 3) Type B Strengthening: including the following
 - a. Strongbacks to increase the out-of-plane resistance of the walls
 - b. In-plane wall strengthening, such as concrete/steel frames or shotcrete overlays

Note that in-practice Type B strengthening is never performed without Type A, so the levels of strengthening would actually be considered 1) Parapet Bracing, 2) Type A, 3) Type A+B.

Ingham & Griffith (2011b) provides a breakdown of the buildings by retrofit status for the 370 buildings in the CBD as follows:

- 139 of 370 (38%) CBD buildings have no confirmed strengthening
- 149 of 370 (40%) CBD buildings have Type A strengthening
- 82 of 370 (22%) CBD buildings have Type A+B strengthening

Statistics on parapets were provided for each parapet, rather than for each building. They were as follows:

- 149 of 435 (34%) parapets were braced
- 89 of 435 (21%) parapets were confirmed as not being braced
- 197 of 435 (45%) parapets had unknown bracing statuses

Neither of the Ingham and Griffith reports (2011a, 2011b) attempted to correlate the damage with ground motion intensity. Instead, overall statistics for the usability placards, ATC-13 damage states, and component-specific damage assessments are provided. The following sections summarize the data. Dr. Jason Ingham and his fellow researchers graciously provided the author with the database collected. Further analysis was undertaken by the author and is presented in Section 4.8.4.

4.7.4.2 Placard Data

Placard statistics for the two earthquakes were markedly different. For the September earthquake, a sample of 596 buildings was analyzed, showing that 47% were tagged green, 32% were tagged yellow, and 21% were tagged red (Ingham and Griffith 2011a). For the February earthquake, the sample of 370 CBD buildings contained 1% green, 17% yellow, and 82% red tags. Figure 4.17 illustrates the tagging data. No breakdown by retrofitting was provided.

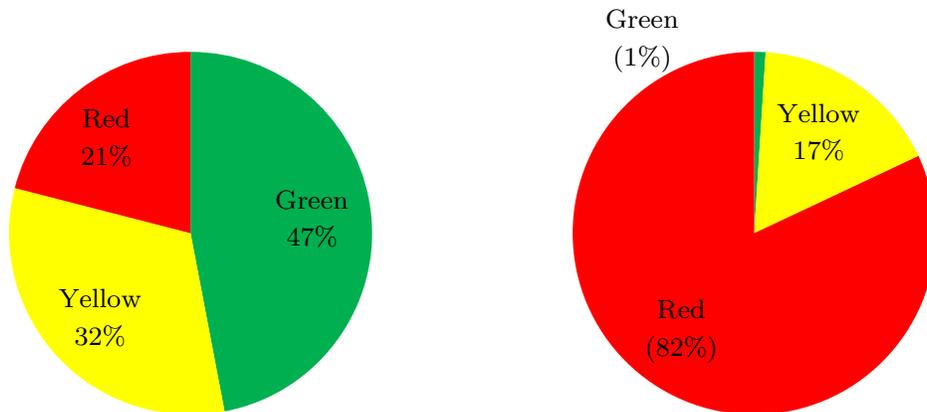


Figure 4.17 – Placard Statistics For September (Left) and February (Right)

From: Ingham & Griffith 2011a, 2011b

4.7.4.3 ATC-13 Damage Statistics

As aforementioned, Ingham & Griffith (2011a, 2011b) provided overall damage statistics for each of the two earthquakes. The results for September and February were as shown in Figure 4.18. The February earthquake clearly resulted in much greater damage.

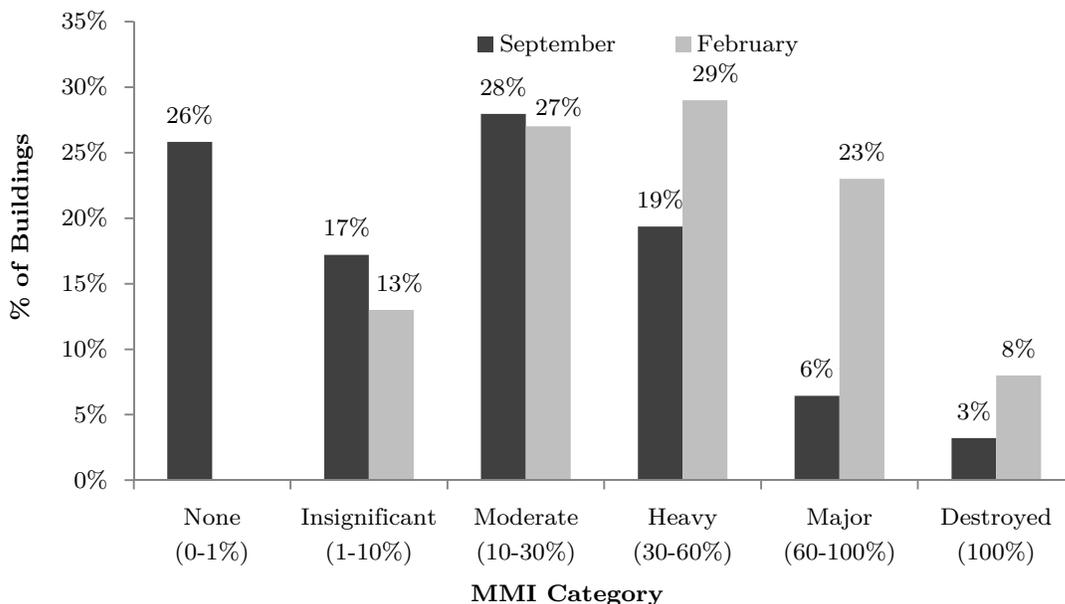


Figure 4.18 – ATC-13 Damage Statistics For September and February

From: Ingham & Griffith 2011a, 2011b

Note that the ATC-13 damage classification was slightly modified: there are only six damage states, instead of seven. In the original document, the 'Slight (1%)' damage state is effectively omitted. However, in the figure above and in the author's subsequent workings with the database, it was assumed that the 'None' and 'Slight' damage states were combined to produce a damage range of 0-1%. Ultimately, this has little effect on the damage statistics.

Although not computed in the original documents, the mean damage factors (computed using the geometric damage factors) associated with the September and February earthquakes would be 22% and 43%, respectively. At first inspection this seemed rather high compared to the California data. However, a review of the recorded PGA's (0.3-0.4g for September and 0.7-1.2g for February) indicates that shaking could perhaps have been closer to MMI VIII, and IX for September and February, respectively (note that this issue was avoided in the author's subsequent work, as MMI was not used). A comparison between Loma Prieta and the September events on this basis is at least reasonable, with an MDF of about 11% for the Loma Prieta buildings (see Table 4.8) versus the value of 23% for the September (Darfield) event. This remaining discrepancy is taken primarily as an example of the relative vulnerabilities of the building stock. The fact that the Canterbury buildings appear more vulnerable is expected given the lack of tension ties in original construction (compared to San Francisco), and the prevalence of cavity walls, two-wythe walls, and gables (Ingham and Griffith 2011a).

4.7.4.4 Damageability Effects of Specific Building Characteristics

The original report (Ingham and Griffith 2011b) also investigates the effect on damageability of various building characteristics. The overall ATC-13 statistics from the 370 Christchurch CBD buildings in February earthquake were the basis for the investigation, including characteristics as noted below. Note that the plots provided were reproduced directly from the original report.

- 1) Number of storeys
- 2) Row vs. Isolated buildings
- 3) Mid-row vs. End buildings
- 4) Building Typology (as defined in Ingham and Griffith 2011a)

The resulting damage appeared to be independent of the number of storeys, as shown by Figure 4.19. Although not calculated in the original document, the resulting MDFs are shown on the figure. Recall that previous studies from the Loma Prieta and Northridge earthquakes showed less than conclusive relationships between the number of storeys and damage, although here there appears to be no trend whatsoever.

A more promising result was obtained with regards to row versus isolated buildings, as shown by Figure 4.20, with isolated buildings suffering more damage. Similarly, buildings at the ends of rows suffered more damage than buildings in the middle. Lizundia et al. (1993) recorded information on adjacencies, although no work was performed as to the effects on damageability.

Finally, a reasonable trend was found with regards to the typology, as shown in Figure 4.21. The typologies are summarized in Table 4.18. Ingham & Griffith state that Type B and D buildings – one and two-storey row buildings, respectively – were shown to suffer less damage. However, calculation of the MDFs reveals that Type A and B buildings were the least damaged.

Table 4.18 – New Zealand URM Typologies

(From: Ingham & Griffith, 2011a)

Type	Description	Details
A	1 Storey, Isolated	One storey URM buildings. Examples include convenience stores in suburban areas, and small offices in a rural town.
B	1 Storey, Row	One storey URM buildings with multiple occupancies, joined with common walls in a row. Typical in main commercial districts.
C	2 Storey, Isolated	Two storey URM buildings, often with an open front. Examples include small cinemas, a professional office in a rural town and post offices.
D	2 Storey, Row	Two storey URM buildings with multiple occupancies, joined with common walls in a row. Typical in commercial districts.
E	3+ Storey, Isolated	Three+ storey URM buildings, for example office buildings in older parts of Auckland and Wellington.
F	3+ Storey, Row	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	Churches (with steeples, bell towers, etc), water towers, chimneys, warehouses. Prevalent throughout New Zealand.

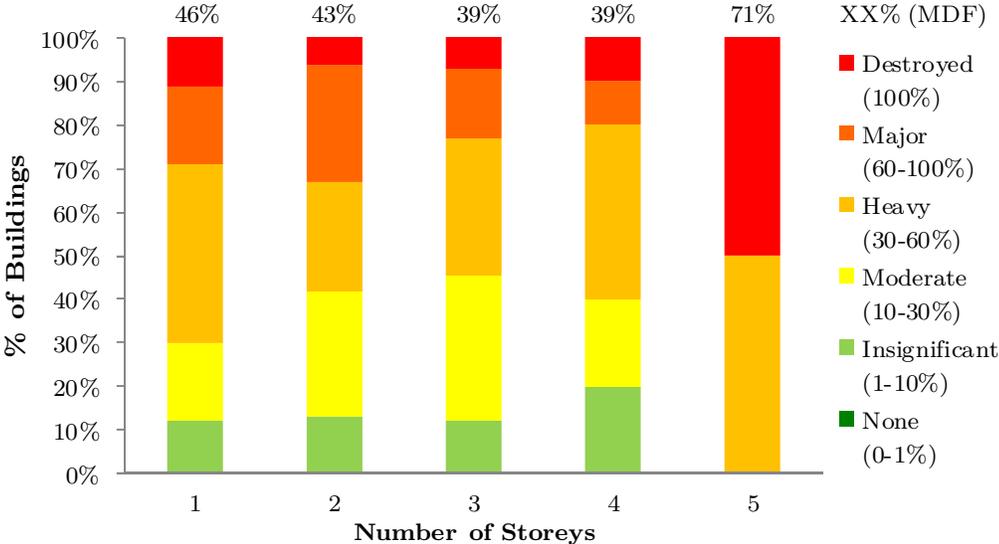


Figure 4.19 – Damage vs. Number of Storeys
From: Ingham & Griffith 2011b

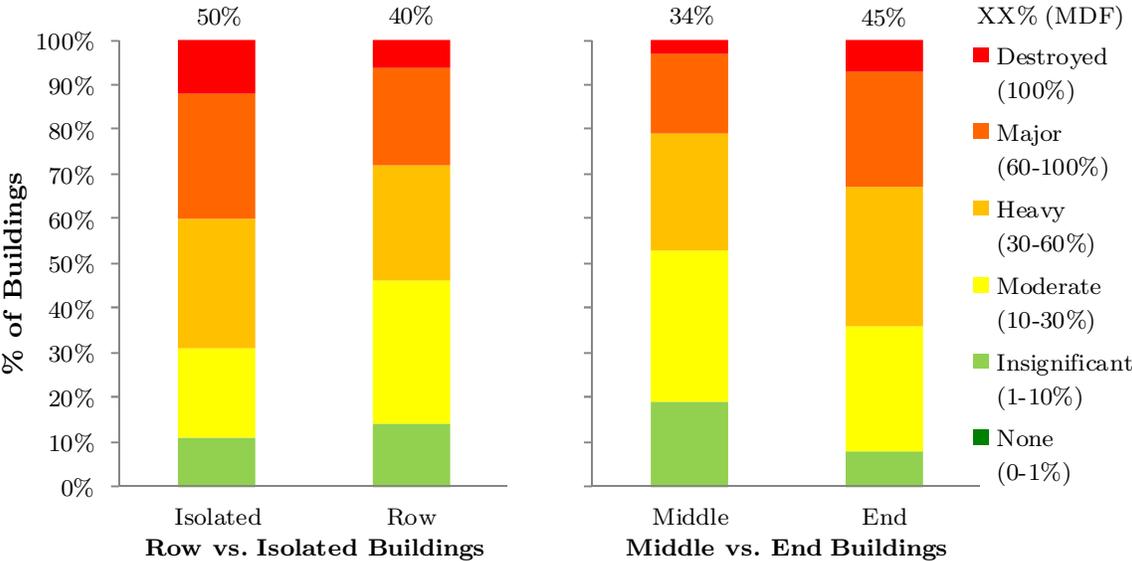


Figure 4.20 – Damage for Row vs. Isolated (Left) and Middle vs. End (Right)
From: Ingham & Griffith 2011b

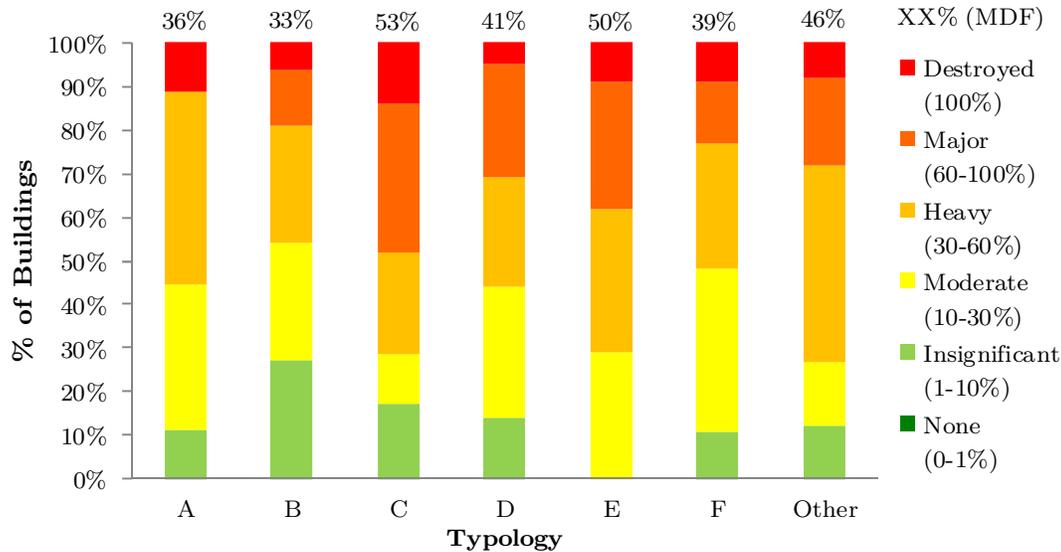


Figure 4.21 – Damage by Building Typology

From: Ingham & Griffith 2011b

Note that ‘Other’ is assumed to essentially represent the Type G (institutional, religious, industrial) buildings. It is interesting that the statistics indicate that 1-storey buildings as a whole suffer no less damage than others, yet the typology statistics indicate that the typical commercial 1-storey buildings (row or isolated) suffer less damage. A brief review of the database indicated that a number of Type G buildings were also one-storey buildings, which is thought to be the explanation for this observation. The review also indicated that the sample of Type A buildings was quite small (just 9 buildings), further supporting this theory. In comparing this result to the Loma Prieta study by Lizundia et al. (1993), it should be noted that 1-storey buildings had a lower MDF than 2- and 3-storey buildings here as well (3.77%, as compared to 4.85% and 4.68%). For a better comparison, a breakdown of the Loma Prieta buildings by the New Zealand typology was undertaken by the author and is presented in Section 4.8.2.3.

One further observation based on the MDFs is that Type C and E buildings (two and three storey isolated buildings) appear to suffer similar damage. The same can be said of Type D and F buildings (two and three storey row buildings).

4.8 New Damage Statistics Results

As previously noted, the fact that there is no data on the observed seismic performance of western Canadian URM buildings provided motivation to review a number of other building populations, so as to obtain a reasonable average. This led to the extensive literature review, as summarized in Section 4.7. However, there were limitations in the results as presented by others (eg. the use of MMI) as well as opportunities for further investigation (eg. the limited statistical analyses of the Canterbury data). In this section, the following further studies completed by the author are presented:

- Converted damage statistics to a common format. Results are presented in terms of DPMs and MDF plots with $S_a(1)$ as the ground motion intensity measure
- Analyzed the Loma Prieta database to investigate the performance of buildings with braced parapets
- Analyzed the Loma Prieta database to further investigate the damageability effects of various building characteristics as defined for the Canterbury buildings
- Analyzed the Christchurch database to develop DPMs and MDF plots consistent with the other three databases
- Having been converted to a consistent format, the results of the databases are compared to each other and to published sources for various levels of strengthening
- Developed HAZUS-compatible damage state structural fragilities for URM buildings for each strengthening level based on the observed MDF vs. $S_a(1)$ relationships

4.8.1 Whittier 1987 Earthquake

The Whittier earthquake was the only earthquake of the four for which a database was not available. As such, all that could be done was to convert the ground motion intensity values from MMI to the chosen ground motion parameter, $S_a(1)$. This was performed in a two-step process.

4.8.1.1 Converting MMI to $S_a(1)$

First, the MMI measurements were converted to Peak Ground Velocity values, based on the relationship established by Wald et al. (1999). Note that the relationship was established by regression analysis for eight California earthquakes, including Whittier, Loma Prieta, and Northridge. The corresponding values for MMI (or Instrumental Intensity as it is referred to for these purposes) and PGV are shown in the USGS-generated shakemaps, such as Figure 4.16. It was assumed that each bin could be represented by its midpoint (eg. for MMI VII, the midpoint PGV of 23.4cm/s was used).

In the second step, the resulting PGV is converted to $S_a(1)$. It was possible to achieve this with a simple linear regression analysis because PGV and $S_a(1)$ are generally correlated quite well. The regression analysis was performed for each individual earthquake (i.e. Whittier in this case) using the seismic station values as listed in the United States Geological Survey shakemap archives; see the captions of the previously shown shakemaps for web links. In this case, the correlation coefficient between PGV and $S_a(1)$ was 0.926, with an R^2 of 0.86. Figure 4.22 provides a plot of the data as well as maps showing the spatial similarities. A more sophisticated model could perhaps have accounted for more of the scatter in the data, but this simple model was deemed accurate enough for the task at hand, which was simply to estimate the expected $S_a(1)$ for a given PGV, which represents a reasonably large range of values and will ultimately be binned quite coarsely. The same process was also completed for other databases discussed shortly, with similar results.

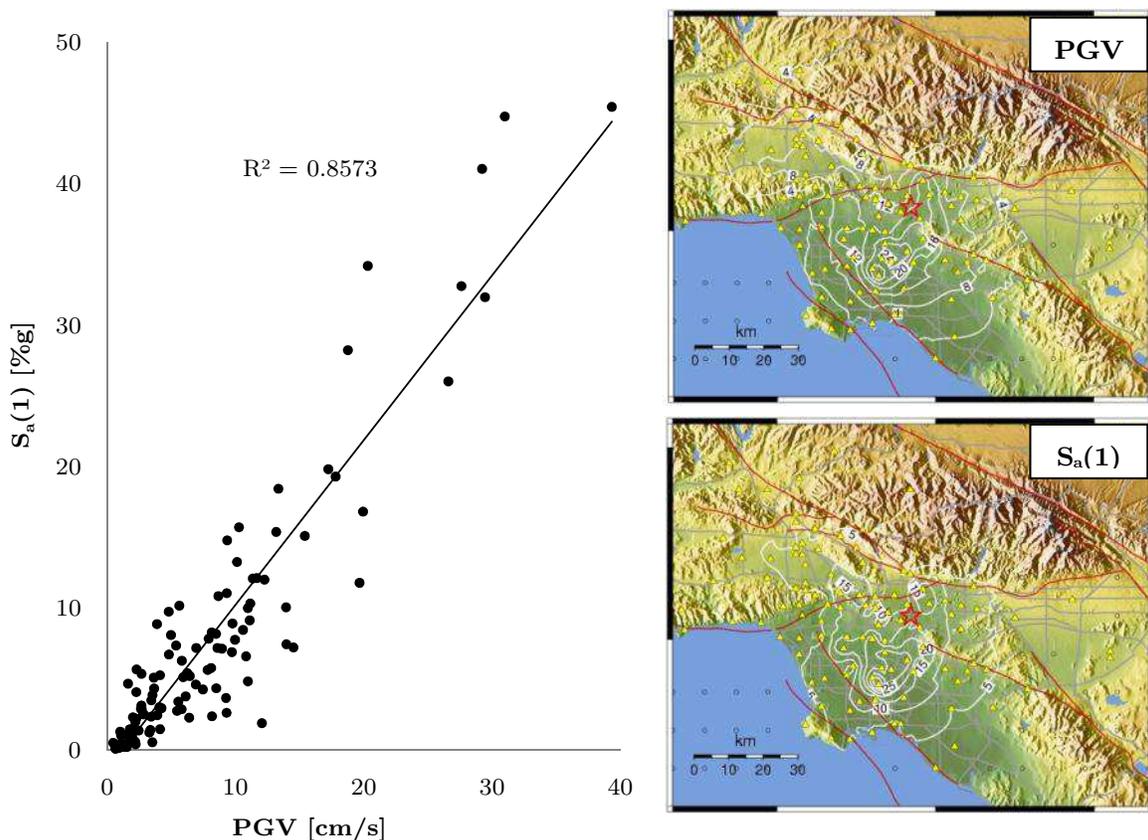


Figure 4.22 – Whittier PGV vs. $S_a(1)$ Regression (left) and USGS Shakemaps (right)

4.8.1.2 Resulting DPMs and MDF Plots

With the MMI values converted to $S_a(1)$, the DPMs can be expressed in terms of $S_a(1)$. This represents just a small change to Tables 4.5 to 4.7 Appendix B contains the DPM.

Since we are ultimately interested in the damage at any point, rather than just the measured intensities, the resulting MDFs were plotted as a function of $S_a(1)$ and a probability distribution was fit to the data. The beta distribution was fitted to the data using nonlinear regression with a weighted least squares criterion, as discussed in Section 4.6. This exercise was performed for each of the three subsets of buildings: unretrofitted, partially retrofitted, and fully retrofitted. Figure 4.23 provides a plot of the results.

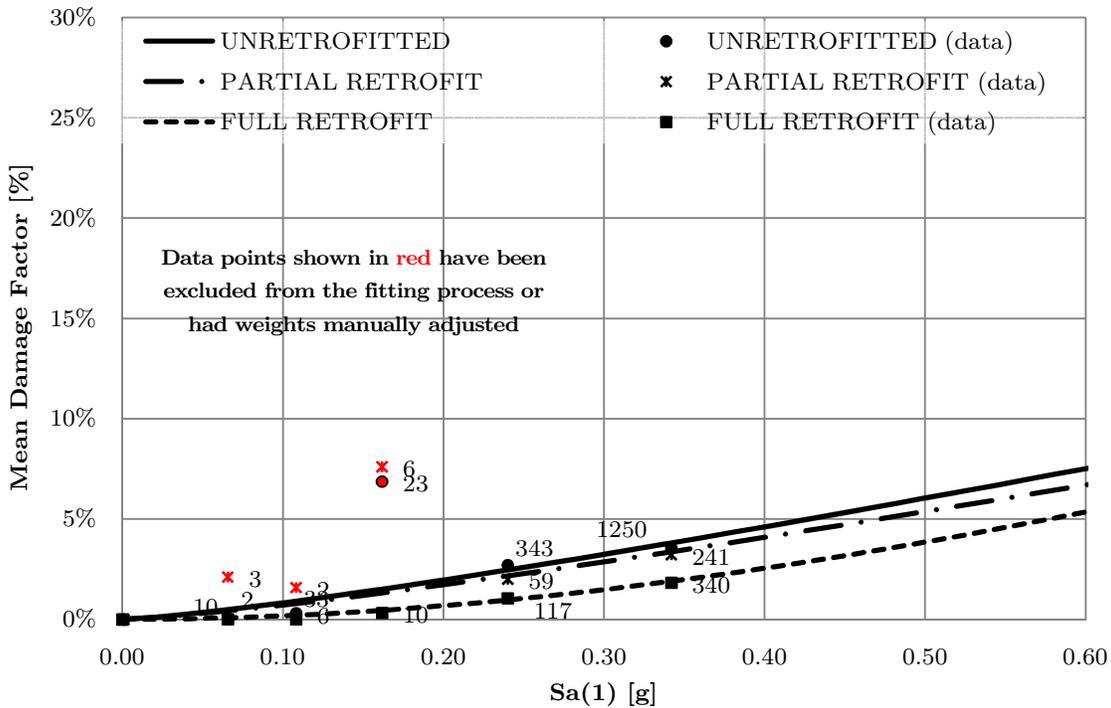


Figure 4.23 – MDF vs. $S_a(1)$ For Whittier Earthquake

The number next to each data point in the graph above represents the number of buildings associated with that point. Note that the red markers represent data points that were excluded or had their weights manually reduced (beyond the typical weighting based on their standard error) because they appeared to be outliers. The cause was usually a bin with just a few buildings, all at the same (or nearly the same) damage state, which would give an erroneous MDF value, but a standard error near zero. In other cases, it may be due to erroneous estimates of ground motion at the site; recall that in this particular damage survey, soil effects were only accounted for on a regional basis.

The results are somewhat unusual in that they show little difference between retrofitted, partially retrofitted, and unretrofitted buildings. Overall, the results from the Whittier database are thought to be the least reliable or useful. This is largely due to the lack of variety in the ground motion intensity and the aforementioned issues with soils.

In the $S_a(1)=0.25-0.35g$ range the results are consistent with expectations, showing some marginal improvements for partial retrofitting (essentially just tension ties) and more significant improvements for full retrofitting. One may have expected a larger difference between the ‘unretrofitted’ and ‘partially retrofitted’ buildings; however, the City of Los Angeles implemented a parapet bracing program in 1949 (Rutherford & Chekene 1997), which means that many of the buildings are likely not truly ‘unretrofitted’. Unfortunately, no discussion of this is provided in the original report (Wiggins, Breall and Reitherman 1994). Nonetheless, the results are valuable because it contains the greatest variety of strengthening among the California databases.

4.8.2 Loma Prieta 1989 Earthquake

The database for the Loma Prieta earthquake was graciously provided to the author by its original owner, Mr. Bret Lizundia of Rutherford + Chekene engineers. Therefore, not only was the original DPM for unstrengthened buildings (see Table 4.8) converted to $S_a(1)$, but the database was also further analyzed to determine the damage to buildings with braced parapets.

4.8.2.1 Converting MMI to $S_a(1)$

For the buildings with unbraced parapets, a DPM using MMI was available in the original document (see Table 4.8) and was thus converted. This task was completed in a manner identical to that described in Section 4.8.1.1. The resulting relationship between MMI and $S_a(1)$ was very similar.

4.8.2.2 Resulting DPMs and MDF Plots

The DPM for buildings with unbraced parapets was readily constructed after converting MMI to $S_a(1)$. See Appendix B for the converted DPM.

The original report by Lizundia (1993) excluded buildings with braced parapets from its analysis, because the scope of the report was essentially *unstrengthened* buildings. For the purposes of this study, however, the performance of buildings with braced parapets was of great interest and so the database was analyzed and DPMs were constructed. Note that this represents only about 1000 buildings in San Francisco, not the entire

database, which was the basis of Lizundia’s work.

Since spectral seismic demands were not included in the database, it was first necessary to determine demands at the site. This was achieved in a manner similar to that outlined by Lizundia (1993), and as described below:

- 1) The four closest stations having the same soil category as the subject building were identified
- 2) The values at the subject building were interpolated from each site using the attenuation equation from Boore, Joyner, and Fumal (1997). Note that Lizundia used Boore and Joyner (1988). In any case, the difference has virtually no effect, since only the relative attenuation is used
- 3) The values inferred from the four stations were weighted by distance. The square of the distance was used, as described in Lizundia (1993). While more scientifically justified methods exist, the differences again tend to be masked by the binning. See Section 4.5.2.4 for a discussion on interpolating demands from stations versus other methods
- 4) The data was binned by $S_a(1)$ in increments of 0.05g, ranging from 0.10g to 0.35g

The DPM was then constructed in the typical manner and the MDF was calculated for each bin. As before, the beta distribution was fitted to the data. Figure 4.24 provides the resulting curves of MDF versus $S_a(1)$ for unretrofitted and braced parapet buildings.

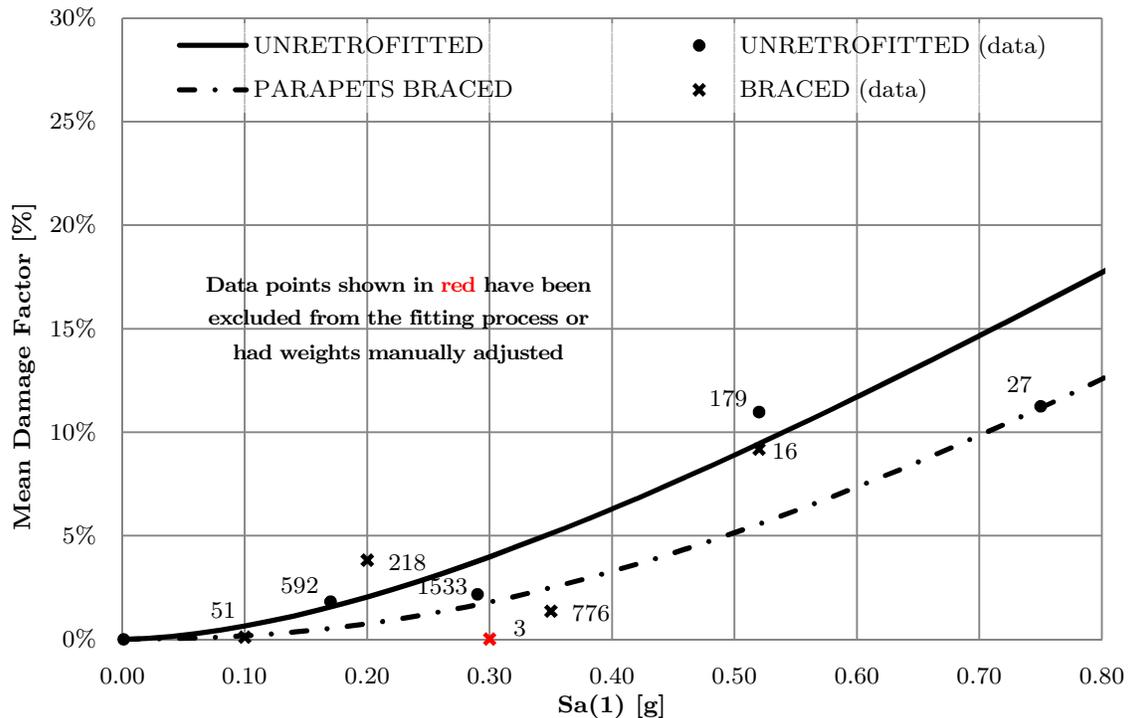


Figure 4.24 – MDF vs. $S_a(1)$ For Loma Prieta Earthquake

Again, the red marker represents an outlier that was excluded. The blue marker was originally, plotted at 0.25g. As there was a reasonable number of buildings and the MDF value was so much greater than expected at 0.25g, it was decided to investigate the demands on these buildings. Because soils were largely responsible for variation in San Francisco, the buildings were mapped along with soils. The map in Figure 4.25 shows that these buildings (marked in blue) were located on the poorest soil in the city; while most areas in San Francisco experienced MMI VI or VII level shaking, USGS maps showed that this area experienced up to MMI IX (see Figure 4.26). As such, these buildings were plotted at MMI 8.5, which translated into $S_a(1)=0.52g$.

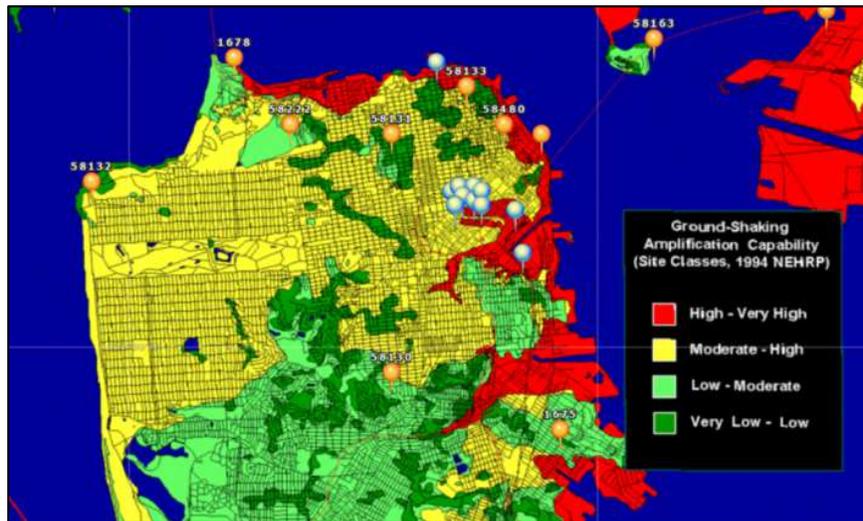


Figure 4.25 – Selected San Francisco Buildings with Soils Conditions

Modified From: http://nsmf.wr.usgs.gov/Presentations/EGSItSFBR/640/Pres3_4.html

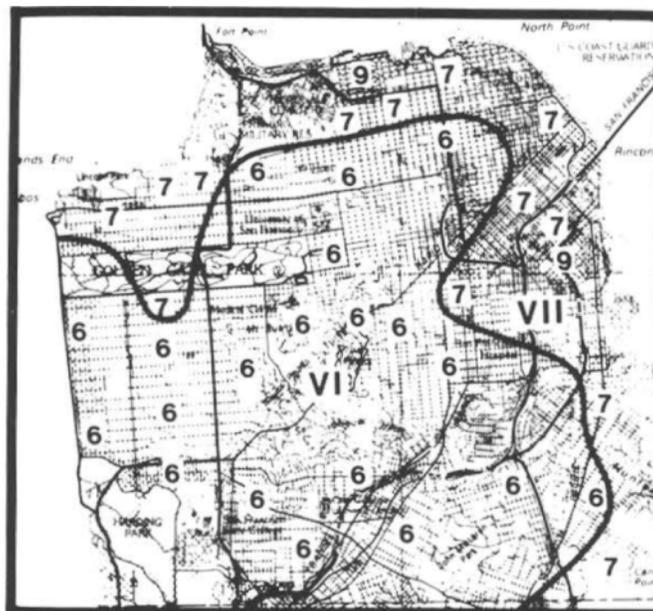


Figure 4.26 – MMI Map of San Francisco (From USGS, 1989)

4.8.2.3 Damageability Effects of Specific Building Characteristics

The original study by Lizundia (1993) looked at the damageability effects of various characteristics. However, as discussed in Section 4.7.2.4 many of these results were not entirely convincing. A few items investigated by Ingham and Griffith (2011b) appeared promising and, as such, those same characteristics were investigated for the San Francisco buildings. The items investigated were as follows:

- 1) Typology
- 2) Middle versus end buildings

4.8.2.3.1 Typology

The first step was to assign the typologies. This was accomplished by making use of several fields in the databases as noted below:

- Prototype: 15 different prototypes, “A” through “O” as specified in Lizundia 1993 (eg. prototype K is two and three storey, small area, residential)
- Building use: Residential, Office, Commercial, Industrial, Assembly
- Number of Storeys
- Adjacencies: provided for all four sides of each building, noting the presence of buildings/alleys/streets

For the purposes of classifying buildings as either row or isolated, only buildings with no buildings on any of the four sides were classified as isolated; all others were classified as row buildings. The types were then assigned based on the following rules:

- 1) Buildings of assembly occupancy (Lizundia Prototype ‘O’) were assigned Type G
- 2) Large area, one-storey buildings (Lizundia Prototype ‘B’) were assigned Type G
- 3) Isolated buildings of industrial occupancy were assigned Type G
- 4) 1-storey isolated buildings not classified as G were assigned Type A
- 5) 1-storey row buildings not classified as G were assigned Type B
- 6) 2-storey isolated buildings not classified as G were assigned Type C
- 7) 2-storey row buildings not classified as G were assigned Type D
- 8) 3-plus storey isolated buildings not classified as G were assigned Type E
- 9) 3-plus storey row buildings not classified as G were assigned Type F

Note that Lizundia et al. (1993) prototypes E and F were also industrial type buildings, but these were not automatically assigned to NZ Type G. Based on the storey height as defined by Lizundia et al. (1993), these buildings were found to be more similar to residential or commercial occupancy buildings with a similar number of storeys; this was somewhat intuitive, since many URM buildings with current-day occupancies of

residential or commercial nature were originally used for industrial purposes. The form of a building (and thus the damageability) is obviously more closely correlated with its original occupancy than its current one. Similarly, all Lizundia et al. (1993) prototype B buildings were assigned NZ Type G, since such buildings are typically of industrial origins. Figure 4.27 shows the resulting distribution of the San Francisco buildings. Statistics from the Christchurch CBD and the Canterbury area (i.e. Ingham & Griffith’s entire database) are also provided⁶.

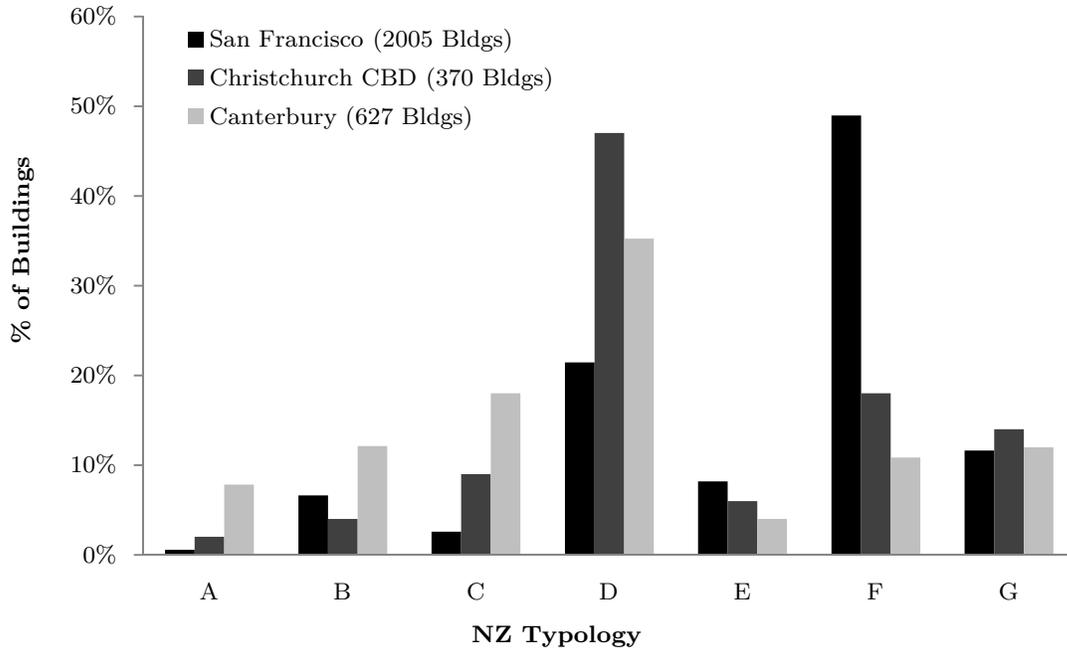


Figure 4.27 – San Francisco vs. Christchurch NZ Typology

Based on the graph above, it appears there exists a greater proportion of 3+ storey buildings in Francisco than in either of the two New Zealand samples. This is at least partly due to the fact that even 5+ storey buildings are somewhat common in San Francisco (326 of 2005 buildings). A review of the Christchurch database showed that only 3 of 627 buildings were of 5+ storeys – instead, two-storey buildings are the most common. Another key difference that is not immediately evident is that isolated buildings are somewhat more common in Canterbury than in San Francisco: Figure 4.28 shows a plot of row versus isolated buildings for the two datasets. This difference is presumably explained by the relative size and density of the two cities, with San Francisco’s population being well over 300,000 by 1900 compared to about 50,000 in Christchurch in 1900.

⁶ Note that "Christchurch CBD" refers to the buildings in the Christchurch CBD while "Canterbury" refers to the entire sample, including buildings in various outlying areas throughout the Canterbury region.

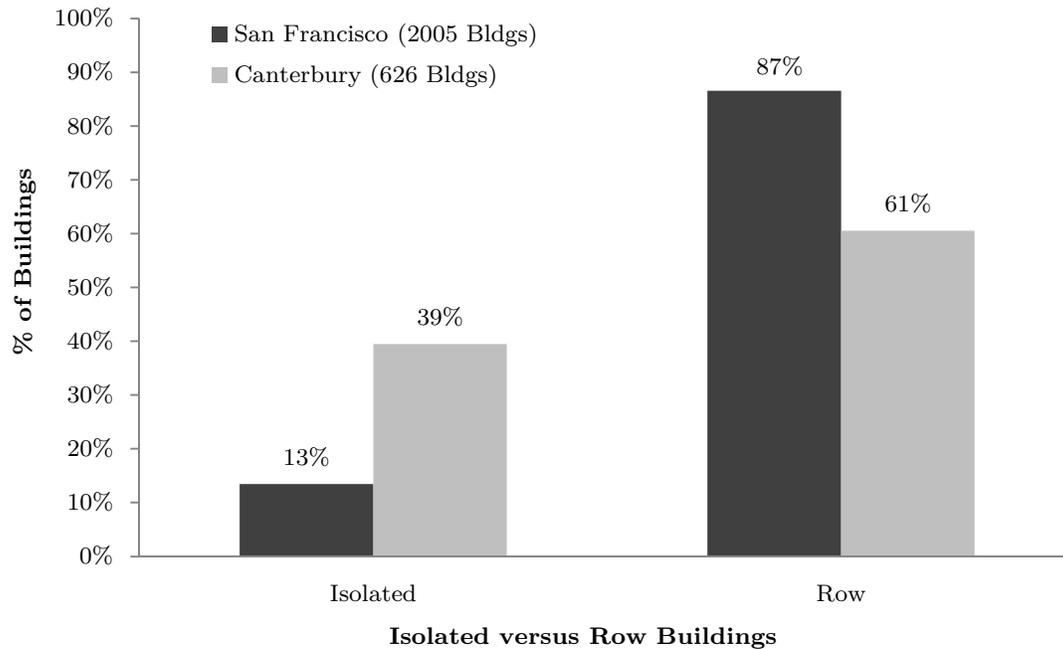


Figure 4.28 – Isolated versus Row Buildings for San Francisco and Canterbury

To further characterize the San Francisco buildings, the entire database of 2005 buildings was analyzed for specific characteristics, by typology. Table 4.19 shows the results. Unfortunately, only a few of these characteristics were recorded in the Christchurch database; however, the available data is shown in Table 4.20; note that this data primarily represents buildings from the CBD.

Table 4.19 – Average Characteristics for San Francisco (2005 buildings)

Type	Total Area [sq.ft.]	Footprint Area [sq.ft.]	Diaph. Aspect Ratio	Storey Height [ft]	# Reentr. Corners	% Soft Storey	% Parapets Braced
A	1636	1636	1.32	17.2	0.30	40%	20%
B	3065	2671	2.73	16.5	0.13	46%	41%
C	16330	8045	2.19	14.4	1.52	27%	25%
D	12097	5896	2.50	13.8	0.29	41%	43%
E	31774	8013	2.19	12.7	1.93	21%	52%
F	20420	4917	2.44	12.6	0.97	44%	70%
G	12486	9653	2.01	18.1	0.74	30%	27%

Table 4.20 – Avg. Characteristics for Christchurch

Type	Footprint Area [sq.ft.]*	Diaph. Aspect Ratio**	% Parapets Braced***
A	3010	2.05	10%
B	4317	2.15	31%
C	3664	2.41	14%
D	2669	2.29	33%
E	3656	2.20	26%
F	4071	2.25	46%
G	7291	2.21	9%

* Based on partial sample of 354 buildings

** Based on partial sample of 331 buildings

*** Based on partial sample of 410 buildings

With regards to the San Francisco buildings, the following observations can be made:

- Type A, B, and G buildings have the greatest average storey heights (the values reported in the original database are the height of the building divided by the number of storeys)
- Row buildings tend to have more elongated diaphragms (i.e. greater “horizontal aspect ratios” as discussed in Section 4.7.3.3) than isolated buildings
- Isolated buildings tend to have more re-entrant corners (the values shown are the average number per building)
- Row buildings have a higher frequency of ‘soft storeys’; this presumably represents open fronts
- Row buildings had a higher frequency of parapet bracing; this is probably because they more frequently had parapets that required bracing under the city’s ordinance

In comparing the San Francisco buildings to the Christchurch buildings, the following observations can be made:

- The Christchurch buildings do not exhibit the same trend for diaphragm ratios
- In terms of footprint area, Type G is again the largest; however, the remaining types are somewhat similar
- The trend among parapet bracing is consistent, with row buildings exhibiting the highest frequency of bracing, and Types A and G buildings exhibiting the lowest

The fact that these results are quite intuitive and reasonably consistent reinforces our confidence that buildings have been assigned correctly to the various typologies.

With regards to damageability, a somewhat different trend was observed for the San Francisco buildings than that found by Ingham and Griffith for the Christchurch buildings: Figure 4.29 shows the ATC-13 damage distribution to San Francisco buildings. Note that for each typology, the MDF is shown at the top and the number of buildings in the sample is shown at the bottom. Only buildings in the $S_a(1)=0.35g$ bin ($\pm 0.05g$) were used, to provide a fair comparison amongst the buildings.

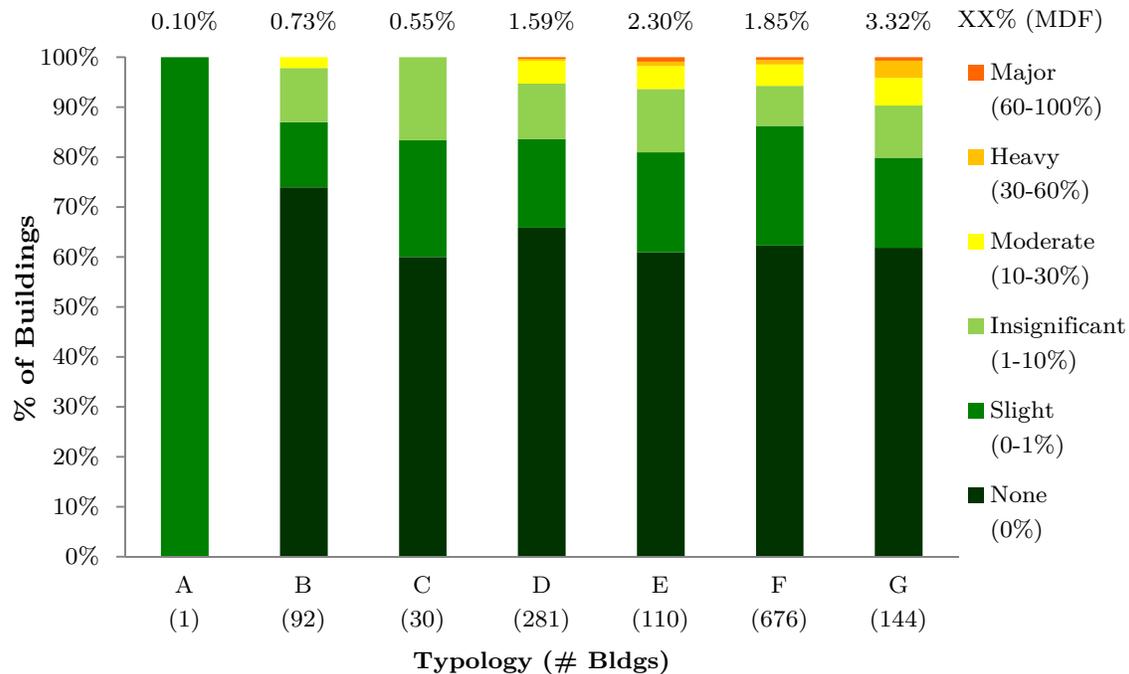


Figure 4.29 – Damage to San Francisco Buildings by Typology

The number of storeys appears to be the more significant factor. For buildings of three or more storeys buildings, row buildings (Type F) suffered less damage than isolated buildings (Type E), similar to the findings of Ingham and Griffith (2011b). However, the trend does not hold for two-storey buildings, although this is perhaps due to the relatively small sample size for Type C. Obviously we cannot draw conclusions about Type A buildings, as only 1 such building was available in the bin. Another observation is that the Type G buildings appear to suffer the most damage. On a relative scale, these differences are more pronounced than those found for the Christchurch buildings. One possible reason is the fact that the intensity of shaking was much higher for the Christchurch earthquake. It is intuitive that the damageability of the buildings would converge at the higher levels of damage (as is often illustrated in fragility curves).

4.8.2.3.2 Middle versus End Buildings

The adjacency information for the San Francisco buildings also enabled the determination of middle buildings, end buildings, and corner buildings. Figure 4.30 shows the relative proportions of middle vs. end buildings for San Francisco as well as Canterbury for comparison. Note that the sum of the middle and end buildings equals 100% since only row buildings are included. As can be seen, the numbers compare well. The database also showed that 22% of the San Francisco row buildings were corner buildings – in other words, about 40% of end buildings were also corner buildings. Unfortunately, this information was not available in the Canterbury database.

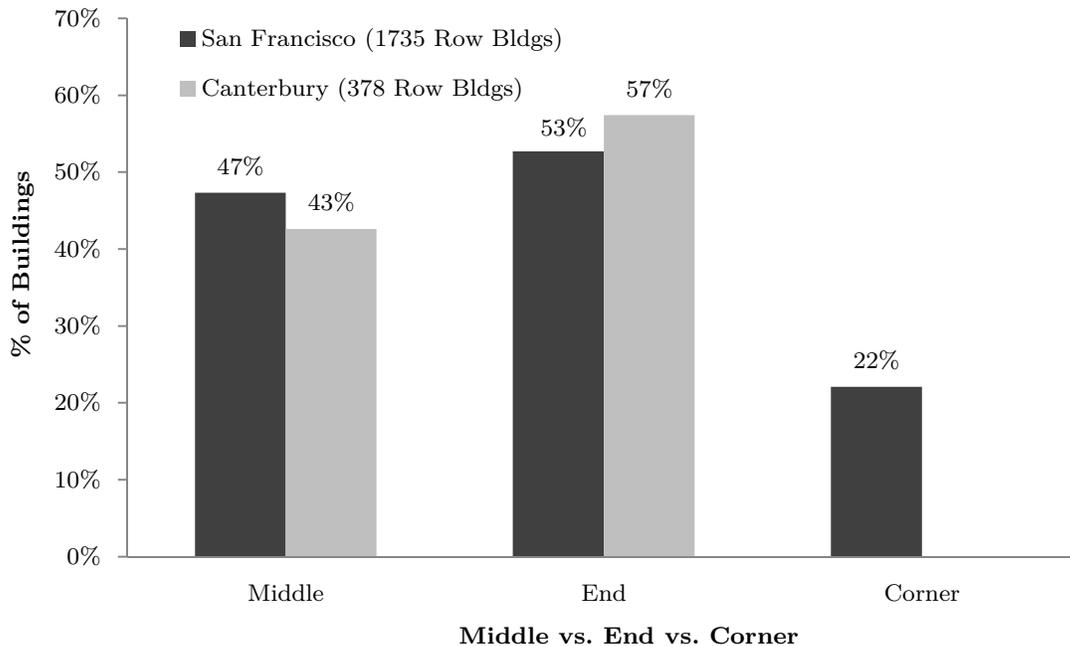


Figure 4.30 – Middle/End/Corner Buildings for San Francisco and Canterbury

With regards to damageability, a similar trend to that for the Canterbury buildings is observed in that end buildings suffered more damage than middle buildings, as shown in Figure 4.31. Interestingly, corner buildings (i.e. buildings on two perpendicular sides) appear to suffer less damage than end buildings (i.e. only one adjacent building). Engineering intuition suggests that corner buildings could be even worse than end buildings due to the eccentricity of the seismic force resisting walls in corner buildings, as the two streetfront walls tend to be highly perforated and therefore much softer and weaker. One possible explanation is perhaps the fact that these buildings have solid walls in two directions more than makes up for the eccentricity effects in terms of global damageability.

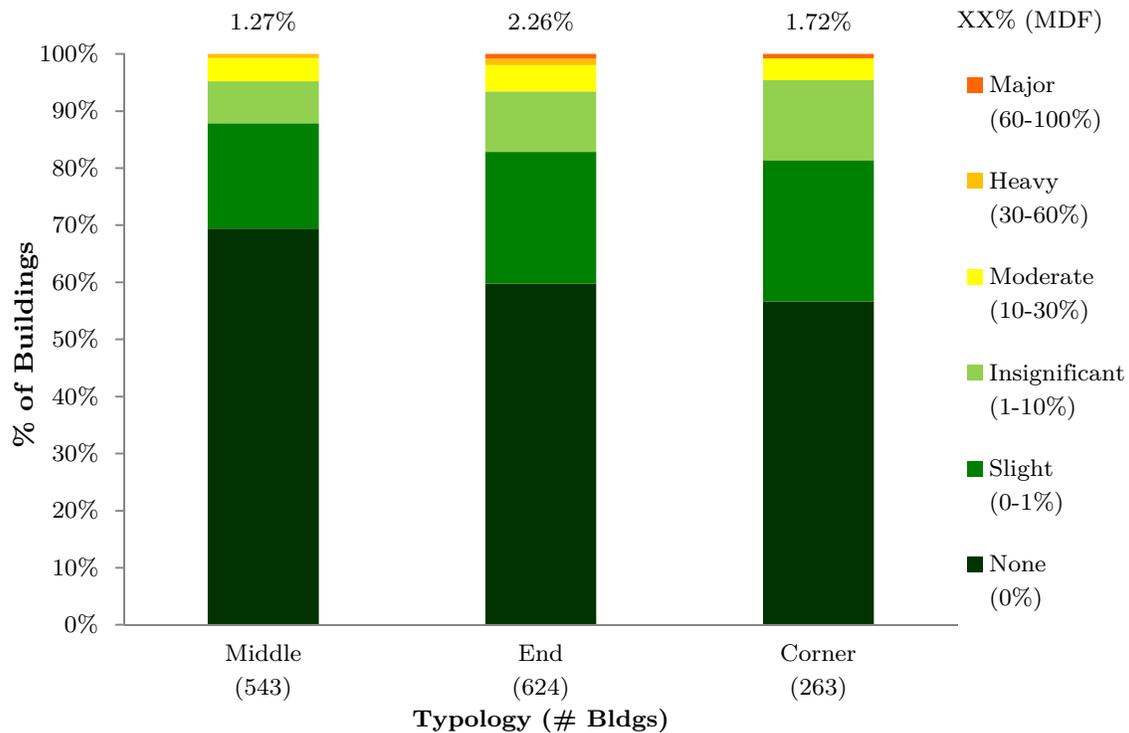


Figure 4.31 – Damage to San Francisco Buildings by End/Middle/Corner

4.8.3 Northridge 1994 Earthquake

This database was again provided to the author by its original owner, Mr. Bret Lizundia of Rutherford + Chekene engineers. As the database was relatively small compared to the Loma Prieta database, no additional investigations were performed.

4.8.3.1 Converting MMI to $S_a(1)$

As the majority of the data was for unretrofitted buildings, only this DPM was converted and used in this study. The task was completed in a manner identical to that described in Section 4.8.1.1. The resulting relationship between MMI and $S_a(1)$ was slightly different, as expected.

4.8.3.2 Resulting DPMs and MDF Plots

The DPM for buildings with unbraced parapets was readily constructed after converting MMI to $S_a(1)$. See Appendix B for the converted DPM.

Again, a beta distribution was fit to the data. Figure 4.32 provides a plot of the results. The results appear reasonable, in that damage is lower than both the unretrofitted and braced parapet buildings from the Loma Prieta database.

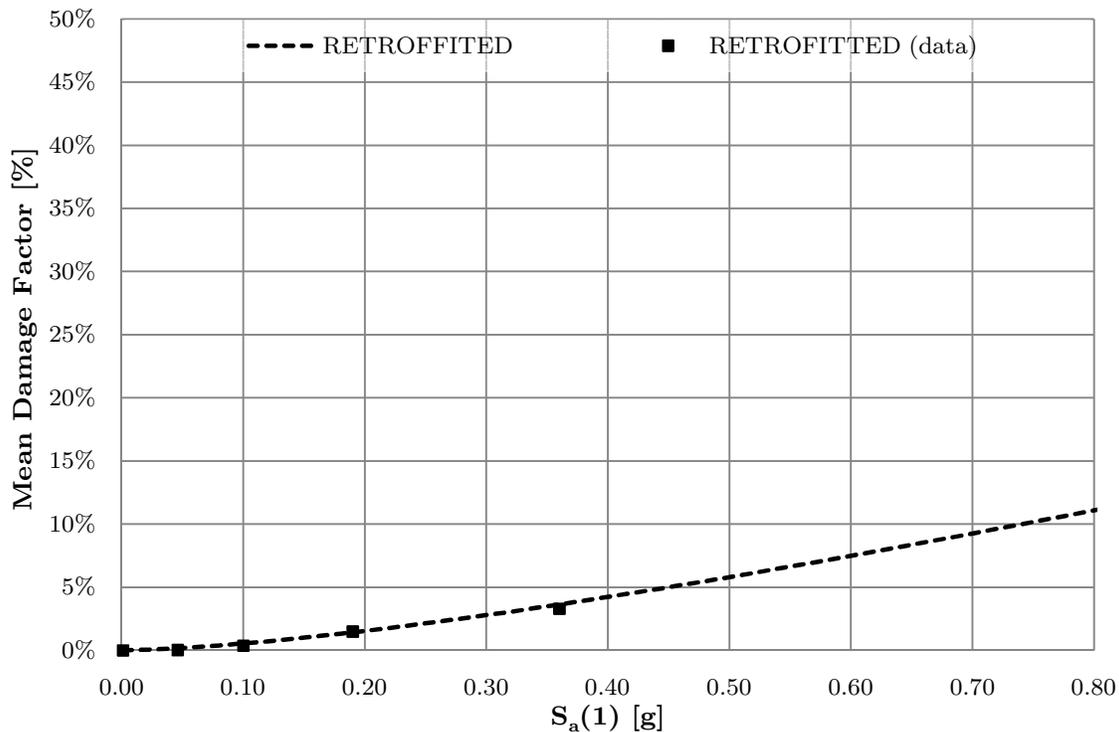


Figure 4.32 – MDF vs. $S_a(1)$ For Northridge Earthquake

4.8.4 Canterbury 2010/2011 Earthquake Swarm

The database for Canterbury buildings was provided by Dr. Jason Ingham and his fellow researchers in New Zealand and Australia. Because Ingham and Griffith's work (2011a, 2011b) primarily examined raw statistics for the 370 Christchurch buildings, plenty of work was completed by the author, which included estimating IM's, constructing the DPM's and fitting distributions to the data.

4.8.4.1 Resulting DPMs and MDF Plots

The first step in constructing the DPM was estimating the ground motions at each site. The ground motion at each site was estimated in the same manner as for the San Francisco buildings (see Section 4.8.2.2).

The DPM was then constructed in the typical manner, the MDF was calculated for each bin, and a beta distribution was fitted to the data. Figure 4.33 provides the resulting curves of MDF versus $S_a(1)$.

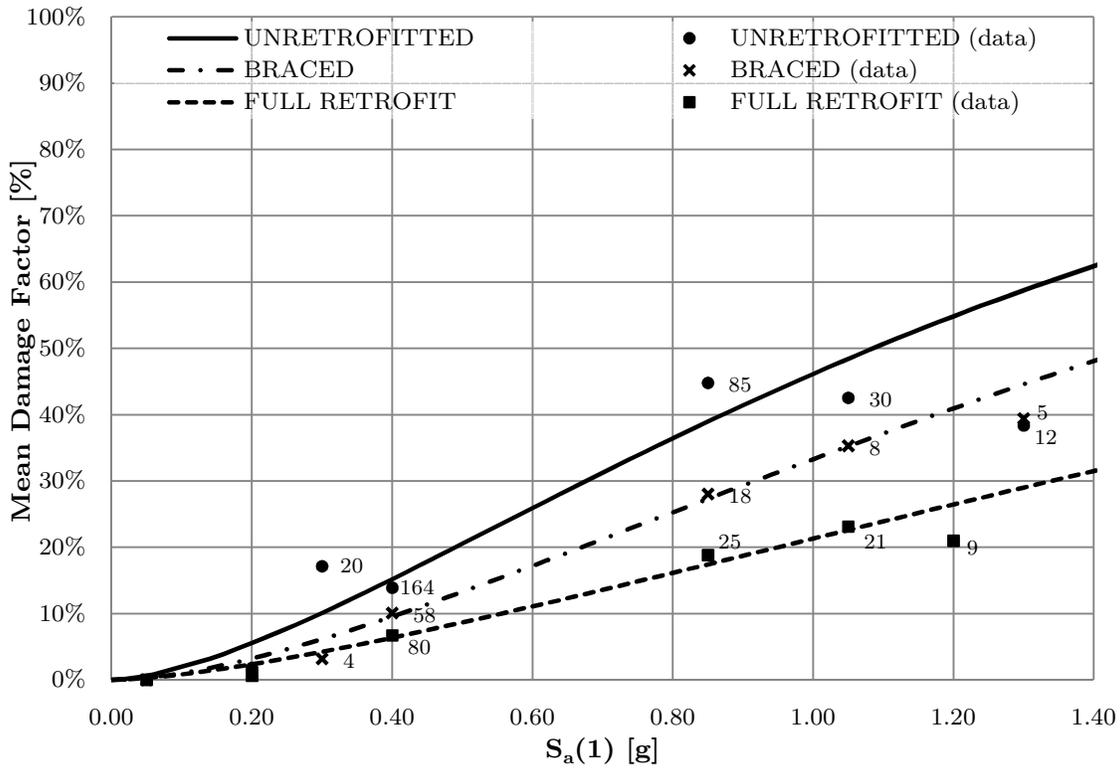


Figure 4.33 – MDF vs. $S_a(1)$ For Canterbury Earthquakes

The results again appear reasonable, with less damage for increased levels of strengthening. Note that “Braced” refers to braced parapet buildings. Clearly, one of the downfalls of the Canterbury database is the relatively small number of buildings. However, it is by far the database with the strongest level of shaking.

4.8.5 Comparison of Results from North America and New Zealand

Having generated results for each of the various databases on common terms, the performance of the buildings can be compared. The following sections compare the results and provide discussion and are organized by strengthening status.

4.8.5.1 Unretrofitted Buildings

As would be expected, the unretrofitted buildings suffered the most damage in each set. Figure 4.34 shows a plot comparing the MDF vs. $S_a(1)$ relationship for each data set.

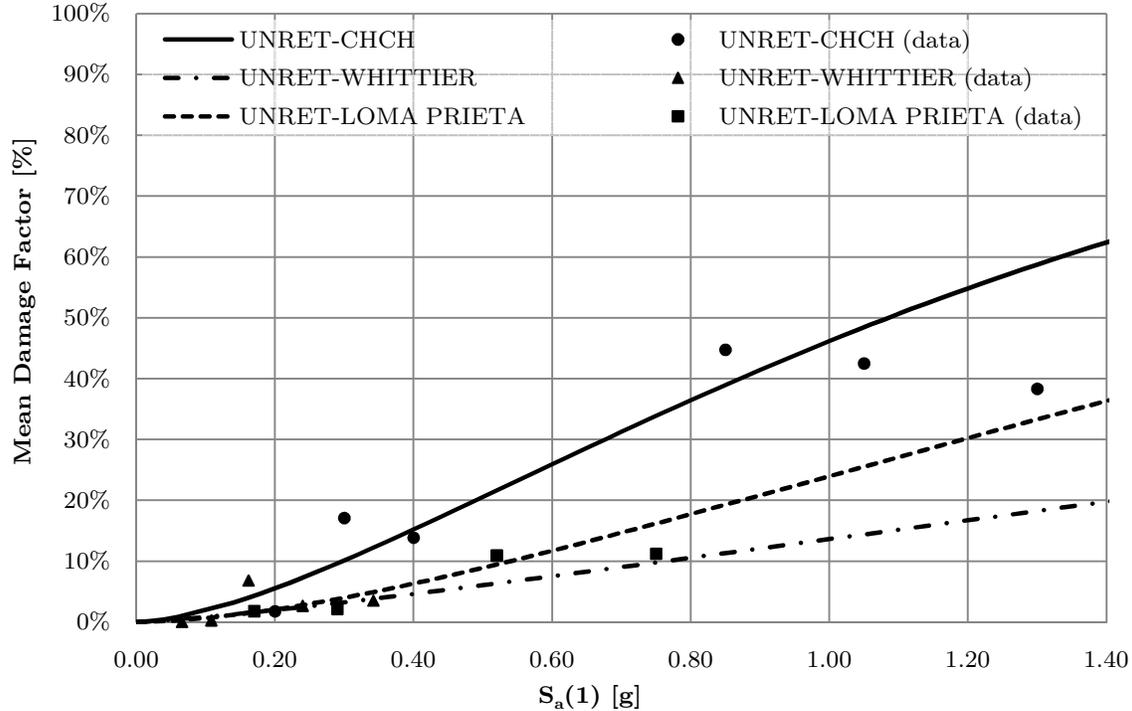


Figure 4.34 – Comparison of Unretrofitted Buildings

A wide range of performance was observed, but this is to be expected given the differing materials, construction practices, and ground motion characteristics. Recall that an unknown, but likely significant, number of the Whittier buildings actually had received some strengthening in the form of parapet bracing. Based on this, combined with the absence of high level shaking, it is postulated that the Whittier data is not an accurate representation of unstrengthened buildings. The Loma Prieta and Canterbury data sets both have their merits and flaws, as previously discussed; however, their results are thought to be much more accurate representations than the Whittier results.

Based on a certain amount of engineering intuition, it is postulated that the Canterbury and Loma Prieta results represent reasonable upper and lower bounds, respectively, on the vulnerability of unretrofitted buildings. The rationale is as follows:

- Several accounts of poor mortar were reported by Ingham and Griffith (2011a, 2011b). Poor mortar quality has also been reported in URM failures in California (Deppe 1988, Lizundia, Dong and Holmes 1993, LATF 1994); however, Lizundia

(1993) reports that the mortar quality in California buildings is fair to good

- Two wythe walls are reportedly somewhat common in the Canterbury region (Derakhshan 2011). While there have been reports of two-wythe walls in California, three-wythe walls are reportedly typical in San Francisco (Rutherford & Chekene 1997). In the author’s experience, two-wythe walls are also uncommon in Victoria, BC, although two-wythe parapets are reasonably common
- Veneer/Cavity wall construction was found to be somewhat common in the Canterbury region: Ingham and Griffith (2011b) report that cavity wall construction (i.e. a gap between outer wythe and inner wythes) was encountered in “almost half of the URM buildings surveyed in Christchurch;” conversely, cavity wall construction is reportedly quite rare in California (Lizundia, 1993)
- Tension ties as part of original construction were reportedly rare in Canterbury: Ingham and Griffith (2011b) report that only 1.6% (6/370) of Christchurch CBD buildings had ties from original construction. It should be noted that 43% of buildings were identified as having retrofit ‘through ties.’ Historically, it has not been uncommon to see original ties mistakenly reported as retrofit items. Nonetheless, original tension ties (known as ‘government anchors’) were common in California, particularly in San Francisco (Rutherford & Chekene 1990)
- The cumulative effect of damage to the Canterbury buildings from the Darfield earthquake likely acted to increase damage in the February earthquake
- A greater proportion of the Canterbury buildings were shown to be isolated buildings (see Figure 4.28)

4.8.5.2 Buildings with Braced Parapets

As expected, buildings with braced parapets appeared to perform better than completely unretrofitted buildings. Figure 4.35 provides a plot comparing the results from the Loma Prieta and Canterbury buildings. Data points in red were excluded from the fitting process (as discussed in Section 4.8.1.2). It can be seen that there is less variation between the two data sets, which is intuitive. For the same reasons as previously discussed, these damage relationships are thought to be reasonable bounds.

4.8.5.3 Partially Retrofitted Buildings

Samples for partially retrofitted buildings were available for the Whittier and Canterbury databases. Note that partial retrofitting is somewhat loosely defined, but the intent is to include tension ties at all floors as a minimum. Figure 4.36 shows the results for these two data sets.

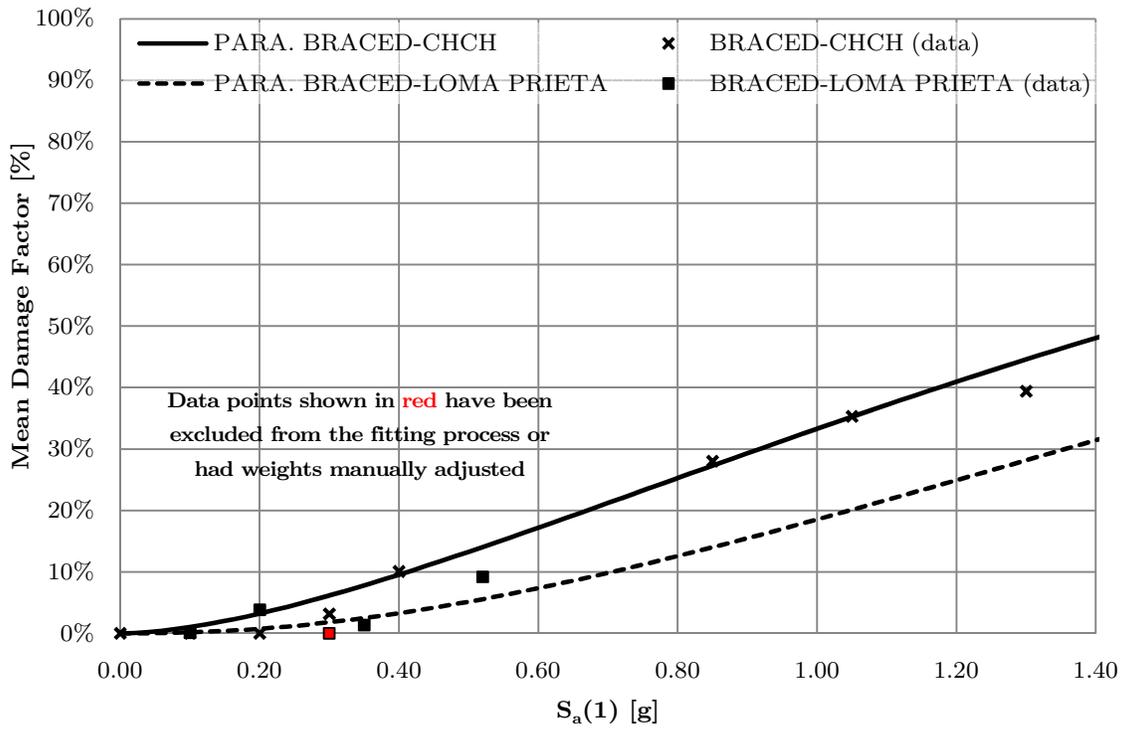


Figure 4.35 – Comparison of Braced Parapet Buildings

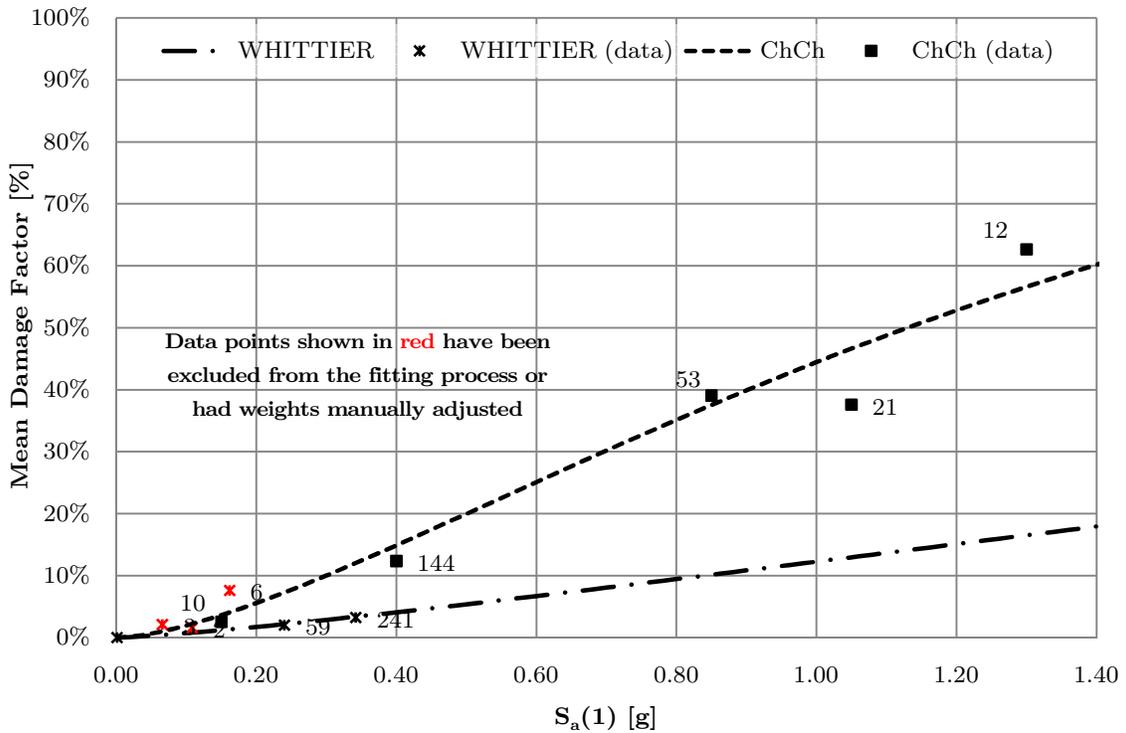


Figure 4.36 – Comparison of Partially Retrofitted Buildings

The results for the partially retrofitted buildings are somewhat disappointing. For the Whittier data, we saw that the partially retrofitted category suffered only slightly less damage. For the Canterbury data, the damage actually exceeded that for buildings that had only parapet bracing (see Figure 4.35). There are many potential explanations for this result: the “partial retrofits” could simply have been poorly conceived. Another issue is the completeness of the scope. For instance, a building could be classified as “partially retrofitted” because one wall had been connected to the floors/roofs, while other walls had not been. If any of the remaining unrestrained walls collapsed, the building performance as a whole would be judged to have been poor.

4.8.5.4 Fully Retrofitted Buildings

Samples for fully retrofitted buildings were available for the Whittier, Northridge, and Canterbury databases (see Figure 4.37). Note that fully retrofitted buildings can be taken to include strengthening measures for out-of-plane (eg. tension/shear ties, strongbacks) and in-plane demands (eg. steel frames, concrete shear walls).

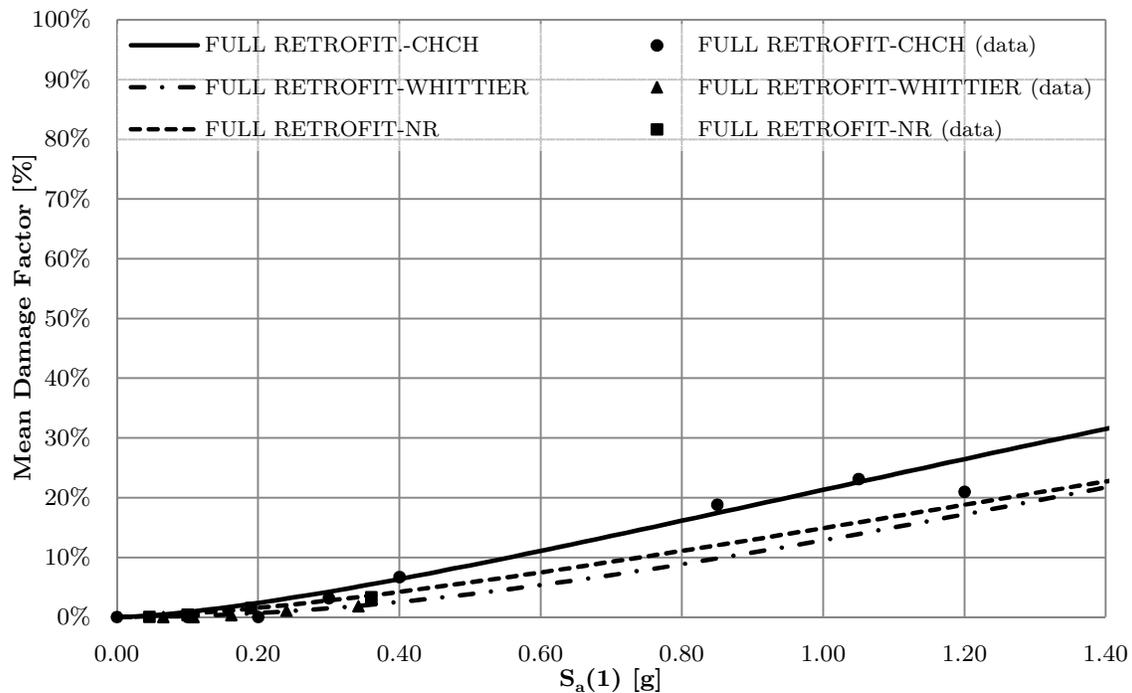


Figure 4.37 – Comparison of Fully Retrofitted Buildings

The results are encouraging in that both damage and variability appear to have been reduced with strengthening. Note that only the Canterbury results include higher levels of shaking. In looking at the Northridge curve, the slope appears somewhat flat at the levels of higher shaking; this may represent an underestimate of damage for the Northridge buildings. Nonetheless, the range of results is relatively small.

4.8.6 Comparison of Results to Published Sources

In an effort to further validate the results, it was decided to make comparisons to published motion-damage relationships. These include:

- 1) “ATC-13” (ATC 1985)
- 2) “Expected Seismic Performance of Buildings” (EERI 1994)
- 3) “HAZUS-MH 2.1” (FEMA 2012)

ATC-13 contains an appropriate relationship for unretrofitted buildings and EERI (1994) contains a relationship for buildings retrofitted to the Uniform Code for Building Conservation, which is essentially the same as current American and Canadian URM retrofit standards. HAZUS contains an appropriate relationship for unretrofitted buildings and, although it does not provide one for strengthened URM buildings, it suggests (in passing) the use of its relationship for low code reinforced masonry in lieu.

4.8.6.1 Unretrofitted Buildings

ATC-13 and HAZUS were both found to contain appropriate motion-damage relationships for unretrofitted buildings. Both sources contained multiple URM types; “Facility Class 75” from ATC-13 and “URMLR (pre-code)” were selected for comparison. Figure 4.38 shows the two relationships along with the relationships from the observed statistics (reproduced from Figure 4.34).

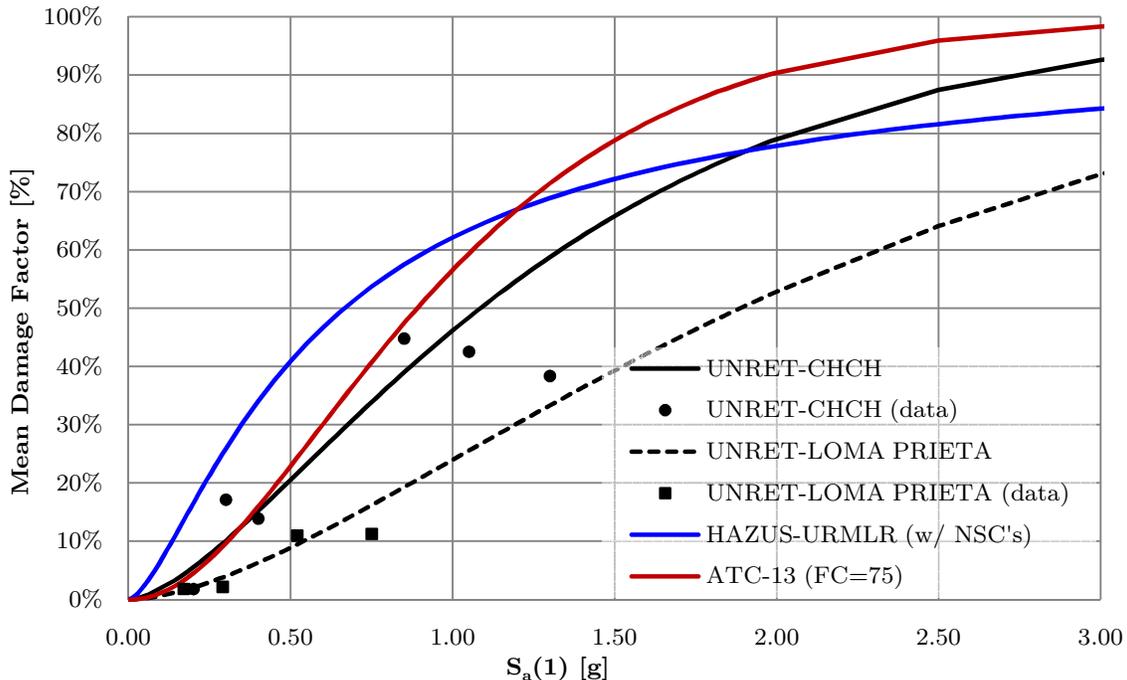


Figure 4.38 – Published vs. Observed Damage for Unretrofitted Buildings

The published relationships are not highly consistent with the observed results, or with one another. At the low-intensity end, ATC-13 is similar to the Canterbury results, but the Loma Prieta results indicate significantly less damage, which was also noted by Lizundia (1993), in his comparison to ATC-13. The HAZUS curve has a somewhat different shape and indicates much higher damage at the low-intensity end. Again, the Whittier results have been excluded, since they are thought to be an inaccurate representation. Also note that the results have been shown to a much higher intensity level than is truly plausible so as to provide a better sense of the relative shapes of the curves. However, the values above $S_a(1)=2g$ are of virtually no consequence for this study as the probability of exceedance is so low as to not affect the subsequent cost-benefit analyses.

4.8.6.2 Retrofitted Buildings

EERI (1994) and HAZUS were both found to contain appropriate motion-damage relationships for retrofitted buildings. As aforementioned, the EERI relationship was specifically developed for rehabilitated URM buildings, while HAZUS simply recommends the relationship for Low Code Reinforced Masonry. Figure 4.39 shows the two published relationships along with the relationships from observed statistics.

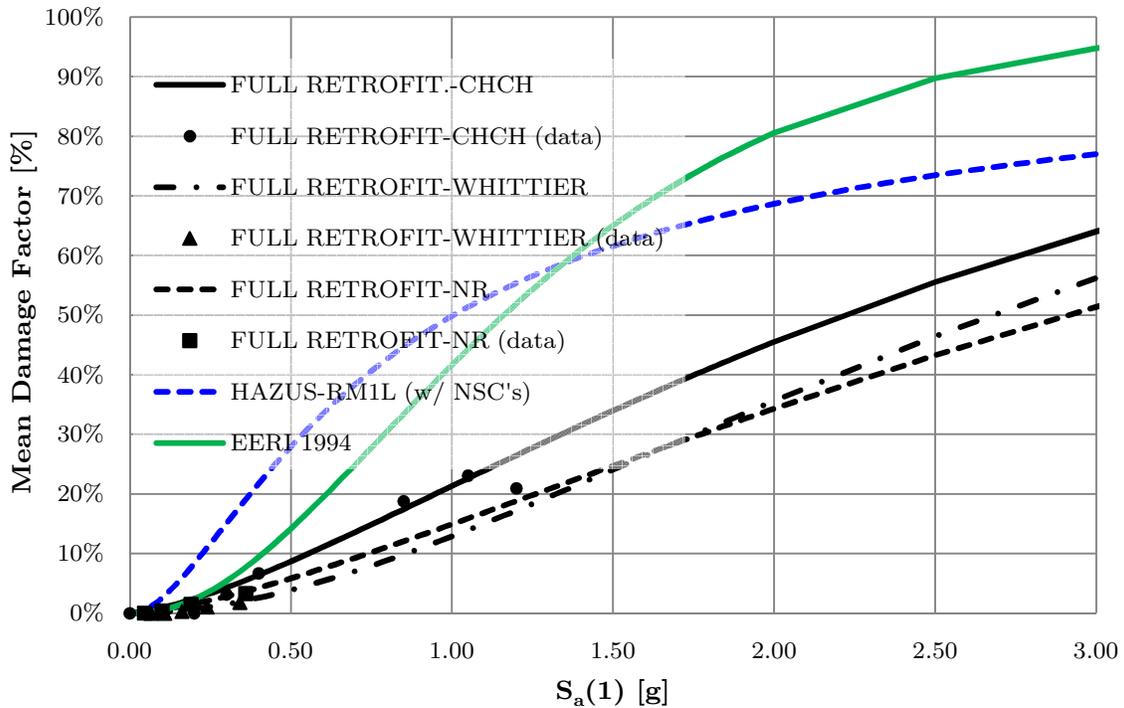


Figure 4.39 – Published vs. Observed Damage for Fully Retrofitted Buildings

For the retrofitted buildings, there is an even greater discrepancy between the published and observed relationships. Rutherford and Chekene (1997) also noted significant differences in their comparison of the Northridge buildings and the EERI relationship.

4.8.6.3 Structural versus Non-Structural Damage

The aforementioned discrepancies between the observed data and published relationships raised questions as to what exactly the published and observed relationships represented. One of the most important distinctions is which building elements are included in the motion-damage relationships. Possible elements include:

- 1) Structural components
- 2) Non-structural components (NSC's)
- 3) Building contents (eg. furniture, computers)

In the case of ATC-13, building elements are simply grouped into two categories: the “facility” (structural and non-structural components) and “equipment” (contents). The previously shown relationship for ATC-13 Facility class 75 (URM Low Rise) is for the facility and therefore includes the non-structural components.

In the case of HAZUS, separate relationships are provided for all three elements, and NSC's are further broken down into acceleration-sensitive and drift-sensitive components. The previously shown relationship includes the structural and all non-structural components and was obtained by combining the three MDF vs. $S_a(1)$ relationships. To combine the relationships, the relative values of the structural and non-structural components must be established. HAZUS provides the following values for a retail trade (COM1) occupancy, which is the most common occupancy for URM buildings in Victoria:

- Structural Value = 29%
- Acceleration Sensitive NSC's = 43%
- Drift Sensitive NSC's = 28%

These values add to 100%, representing the ‘building value’, which is also specified in the document. Contents values are specified as equal to 100% of the *structural* value for this occupancy class and there is also a small value prescribed for business inventory. For the time being however, only the three above noted percentages are important, as these were used to produce a weighted average of the three separate MDF vs. $S_a(1)$ curves, yielding the results shown in Figures 4.38 and 4.39. Of course, the relationship for each element is significantly different. Also note that some work was required to convert the

HAZUS results to $S_a(1)$, since it was developed in terms of spectral displacement.

The significance of the foregoing discussion is appreciated when considering the meaning of the observed damage statistics. In Figure 4.40, the structural-only damage relationships from HAZUS are plotted, along with the relationships developed from the Canterbury data. Both unretrofitted and fully retrofitted curves are presented.

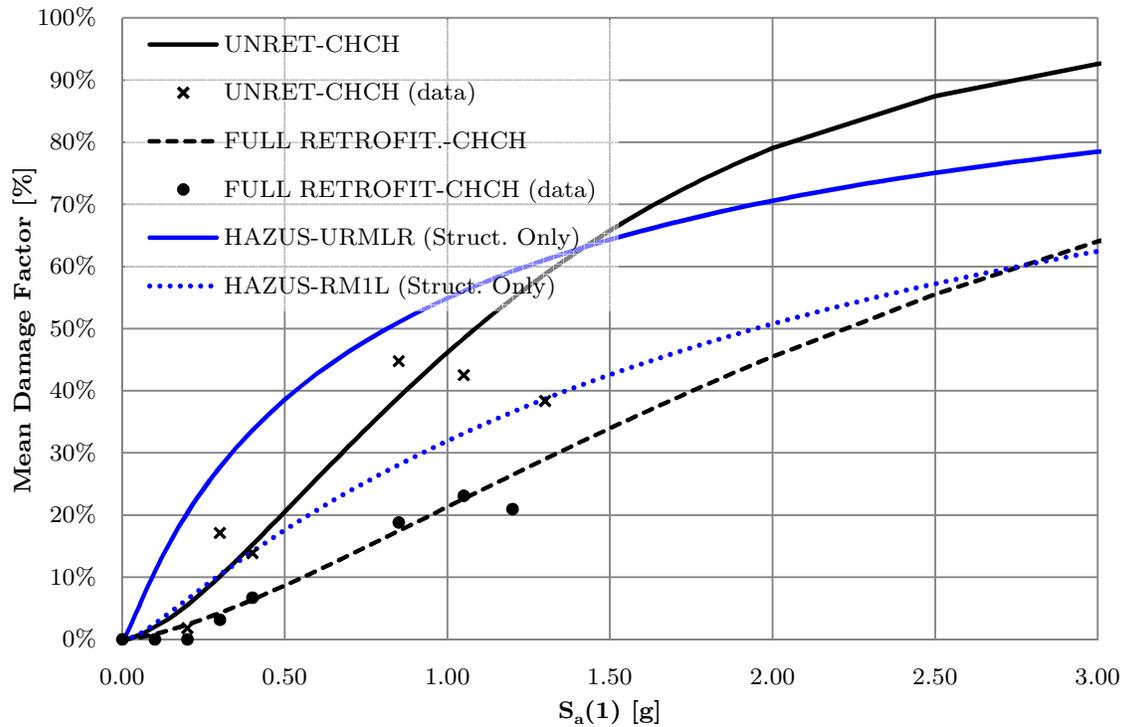


Figure 4.40 – Published vs. Observed Damage for Unretrofitted Buildings

Here, the HAZUS curves are in better agreement with the observed results, although they still generally exceed even the Canterbury data by some margin in the regions of plausible shaking. Although the damage surveys were intended to capture non-structural components, it is easy to rationalize the better fit with structural-only relationships, since post-earthquake evaluations are often performed primarily (if not exclusively) from the exterior. Furthermore, for rapid safety evaluations, the inspectors would instinctively have their attention drawn to structural damage since this is more likely to produce serious life safety threats (eg. due to collapse).

4.9 Motion-Damage Relationships for Victoria

Having completed the extensive review of damage to URM buildings in other regions, the focus is now shifted to Victoria and an attempt to define appropriate motion damage

relationships is made. First, the level of vulnerability for Victoria relative to the other regions is rationalized on a subjective basis and then the curves for various levels of retrofitting are presented.

Based on the discussion of Section 4.8.6.3, it was concluded that the observed data better represented structural-only damage and so the data is used to define curves for the structural-only damage. Motion damage relationships for the non-structural components and building contents will be those as defined in HAZUS.

4.9.1 Relative Vulnerability of Victoria URM Buildings

For reasons discussed in Section 4.8.5, it is postulated that the vulnerability of Victoria buildings lies somewhere between those of the California and Canterbury buildings. Beyond this, there is little that can be said with a high degree of confidence about where the vulnerability of Victoria’s URM buildings should fall. In Table 4.21, we make a subjective comparison of Victoria to the California and Canterbury characteristics as noted in Section 4.8.5. An ‘X’ denotes which of the two regions to which Victoria is more similar. Note that items #1 through #7 are presented in what the author believes to be the order of importance to seismic performance. Additionally, a particular emphasis was placed on the Loma Prieta buildings for the California results, as this was the database that was considered more representative of unretrofitted URM buildings. For retrofitted buildings, the Northridge data is the best available Californian sample.

Table 4.21 – Victoria Buildings vs. Other Regions

Characteristics		California ⁷	Canterbury
1	Prevalence of original tension ties		X
2	Prevalence of Cavity Construction	X	
3	Wall Thickness	X	
4	Mortar Strength	X	
5	Number of Stories		X
6	Cumulative Earthquake Damage		X
7	Prevalence of Row Buildings		X
8	Retrofit Standards	X	

Clearly there are mixed results. See Chapter 7 for a characterization of Victoria buildings. Note that Victoria was classified as more similar to the Canterbury buildings in terms of cumulative earthquake damage because of the potential for a subduction

⁷ Note that "California" refers to the Loma Prieta results (Section 4.8.2) for unretrofitted buildings and to the Northridge results (Section 4.8.3) for retrofitted buildings.

earthquake, which comprises a significant portion of Victoria’s seismic hazard. Such a scenario would be expected to generate a long duration of shaking and many aftershocks. On a deterministic basis, one could rank this higher in order of importance, but again, this represents only a portion of the seismic hazard. Of course, the retrofit standards category is an important consideration for retrofitted buildings. As previously noted, Canadian and American URM retrofit standards are quite similar.

Another issue to be considered in deciding where Victoria’s buildings should fall relative to the various results is that the Canterbury data is the only one with extremely high shaking. Even if intuition suggests that Victoria’s buildings are more similar to the California buildings, it seems imprudent to put more stock in extrapolated results than actual observed damage.

4.9.2 Building Specific Damageability Effects

As seen in earlier sections, attempts have been made to link many specific building characteristics to building damageability. For the most part, the efforts have been either unfruitful or inconsistent among different databases. Moreover, no effort has been put forth to investigate the effect of multiple simultaneous characteristics. However, the breakdowns by New Zealand typology for both the Canterbury and Loma Prieta buildings gave results that were reasonably in line with engineering intuition and can, indirectly, account for the presence of multiple characteristics. Based on those results, the following observations were accounted for:

- Isolated buildings suffered more damage than row buildings
- Type G buildings (industrial or religious types) suffered more damage than others
- Type A and B buildings (1-storey structures) suffered less damage than others

The observations were accounted for by adjusting the base curves shown in Figure 4.41. In order to define the adjustment factors, it was necessary to first designate one of the typologies as the anchor point: upon calculating the weighted averages of the mean damage factors for both datasets, the resulting average MDF compared most closely with the Type F buildings and so this was selected as the anchor.

Another observation was that the differences in damage were not as pronounced for the Canterbury buildings. It was postulated that this was primarily due to the increased level of shaking (i.e. at some point the shaking is intense enough that the differences in vulnerability are insignificant and all the structures sustain heavy damage). As such, it was decided to calculate separate factors at the two different levels of ground motion

that were investigated ($S_a(1)=0.35g$ and $S_a(1)=0.9g$, respectively) and to eliminate any difference at a shaking intensity of 50% greater than that which was observed in Christchurch (i.e. $S_a(1)=1.35g$).

By taking the ratio of the MDF of each typology with Type F for the two databases (shown in Figures 4.21 and 4.29), the following adjustment factors were developed. Table 4.22 shows the resulting adjustment factors to be applied to the base MDF curve. It should be noted that the adjustment factors were limited to a $\pm 50\%$ adjustment so as to limit the effect on the subsequent cost-benefit analysis. The adjustment factors were also reduced where the confidence level (from t-testing) was less than 90%.

Table 4.22 – Typology MDF Adjustment Factors

Typology		$S_a(1)=0.30$	$S_a(1)=0.90g$	$S_a(1)=2.0g$
A	1 storey, Isolated	0.75	0.90	1.00
B	1 storey, Row	0.75	0.90	1.00
C	2 storey, Isolated	1.25	1.10	1.00
D	2 storey, Row	1.00	1.00	1.00
E	3+ storey, Isolated	1.25	1.10	1.00
F	3+ storey, Row	1.00	1.00	1.00
G	Institutional, religious, industrial	1.25	1.10	1.00

It should be noted that the differences in the MDF were indeed significant, particularly for the Loma Prieta data: the MDF's were typically at least one to two standard errors (i.e. standard deviations of the mean) away from one another. Although the testing showed less than 90% confidence in differentiating Types C and E from F, the consistent results between the Loma Prieta and Canterbury data suggest there is in fact a significant difference. These adjustment factors were applied to unretrofitted buildings; reduced factors (eg. 25% increase reduced to 12.5%) were applied to buildings with braced parapets and no factors were applied to further retrofits because presumably the underlying deficiencies would be addressed. The following section presents the “base curves” to which these factors were applied.

4.9.3 Defining Fragility Curves for Victoria

Recognizing the uncertainty in the process, three weighted average cases were defined to represent the vulnerability. Case #2 is considered the best estimate and will be the main focus; Cases #1 and #3 will be used in a sensitivity analysis in Chapter 5.

- 1) Upper Bound: 100% weight on Canterbury buildings
- 2) Best Estimate: 67%/33% weights on Canterbury and California, respectively
- 3) Lower Bound: 50%/50% weights on Canterbury and California, respectively

For unretrofitted and braced-parapet buildings, “California” refers to the Loma Prieta results. For retrofitted buildings, “California” refers to the Northridge results. The partial retrofit classification is the most difficult to accurately define as the scope of work is highly variable. Because no reliable results for partially retrofitted buildings were available from California, this curve was taken as the weighted average of the braced parapet and fully retrofitted buildings, with weights of 67% and 33%, respectively; the rationale was that in most of the data encountered, “partial retrofits” were found to provide limited benefits over braced parapet buildings, but logic dictates some nominal increase in performance must be achieved. Obviously, this decision was highly subjective, but this approach was thought to be better than the alternatives (such as not including a partial retrofits category). Figures 4.41 to 4.43 provide the resulting structural MDF vs. $S_a(1)$ curves for Victoria for the best, upper, and lower estimates.

Note that these curves do not yet account for the damageability effects of specific characteristics as discussed in Sections 4.7 and 4.8. Also, only the mean damage factor is addressed for the time being. The distribution of damage into the various damage states (in the form of fragility curves) is also of interest, but was of secondary importance to the study at hand.

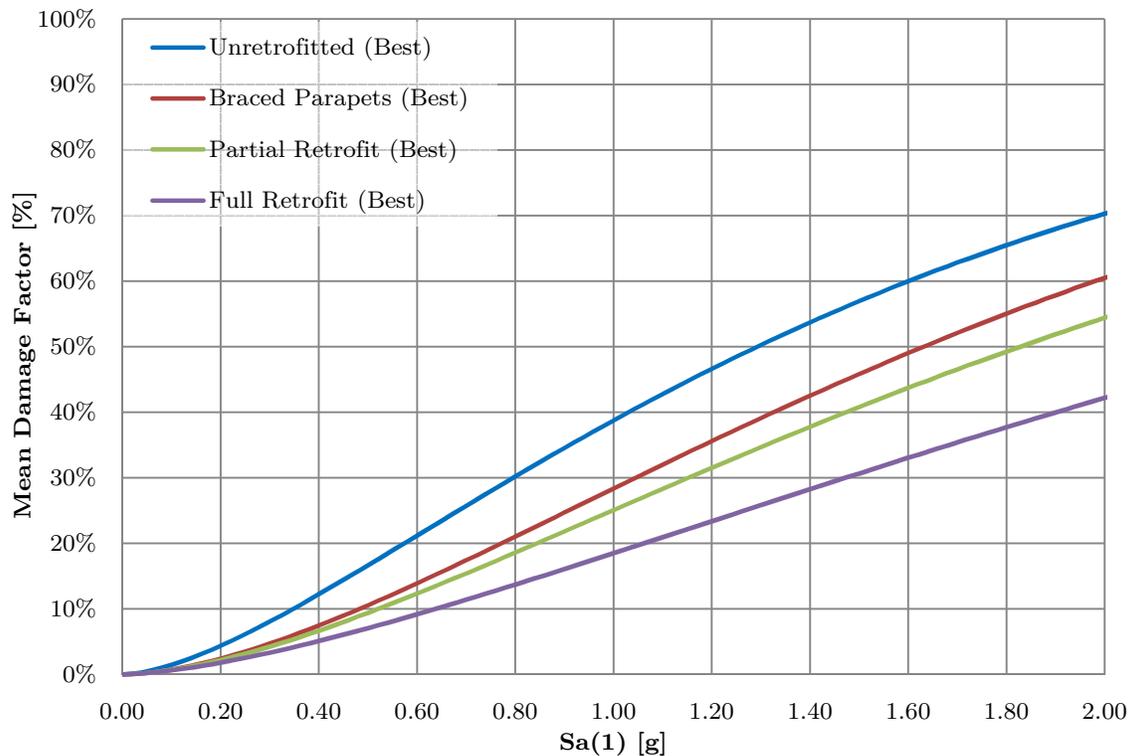


Figure 4.41 – Basic Structural MDF vs. $S_a(1)$ Curves (Best Estimate)

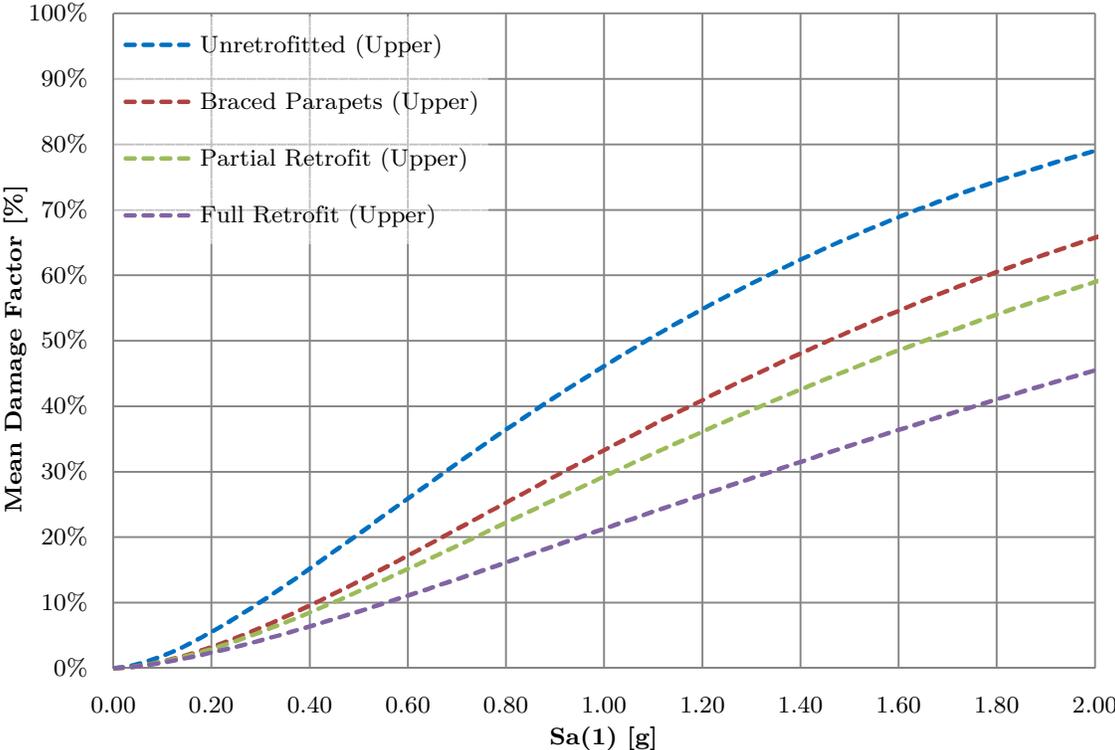


Figure 4.42 – Basic Structural MDF vs. $S_a(1)$ Curves (Upper Bound)

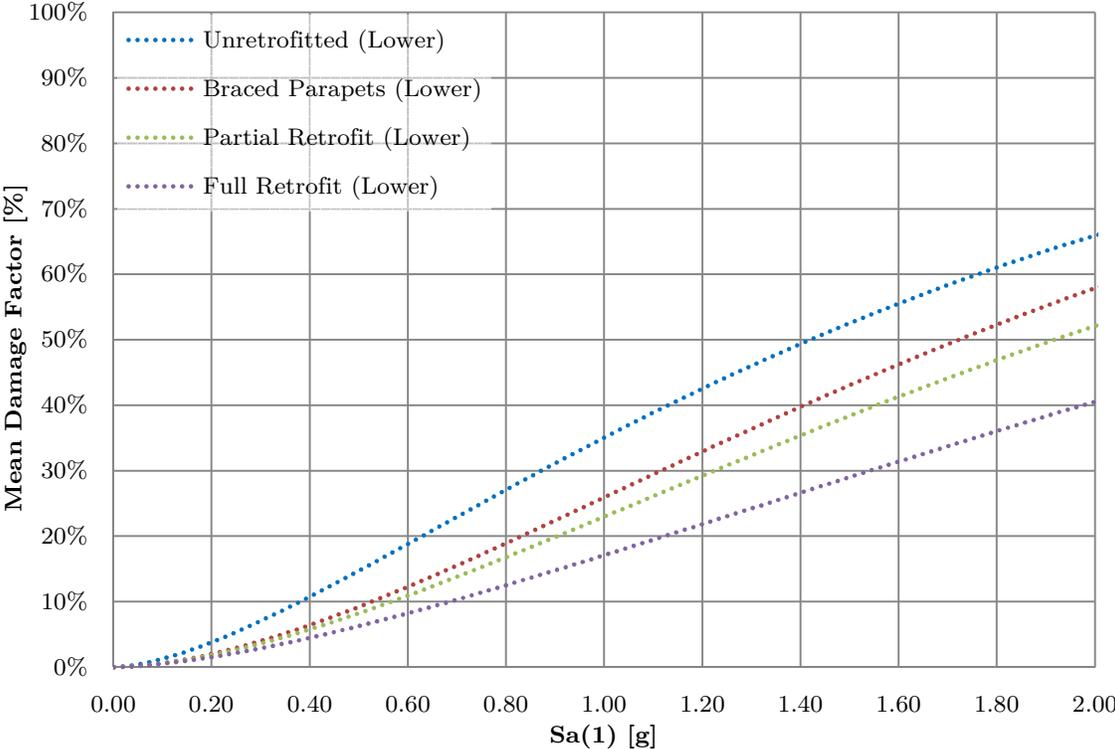


Figure 4.43 – Basic Structural MDF vs. $S_a(1)$ Curves (Lower Bound)

Ultimately, it was decided to not generate the individual damage state fragilities based on the observed data. The reasons are as follows:

- 1) In some cases, there were not a sufficient number of buildings to obtain a statistically reliable estimate for each damage state at every intensity level
- 2) Fitting directly based on data often provides less than satisfactory results, as was the case in even a more focused and sophisticated study by King (2005)
- 3) It was desired that the final motion-damage relationships be compatible with HAZUS, as it is now the standard in earthquake loss estimation. Because all the data was in terms of ATC-13 damage states, it would have been necessary to map the ATC-13 damage states to the HAZUS damage states (through some subjective conversions)

Instead of generating individual damage state fragilities from the observed data, it was decided to generate the fragilities by matching the MDF from our observed results to the MDF associated with the fragilities, while still maintaining a distribution among the damage states that was reasonably similar to the observed data. Figure 4.44 shows an example result for Unretrofitted buildings.

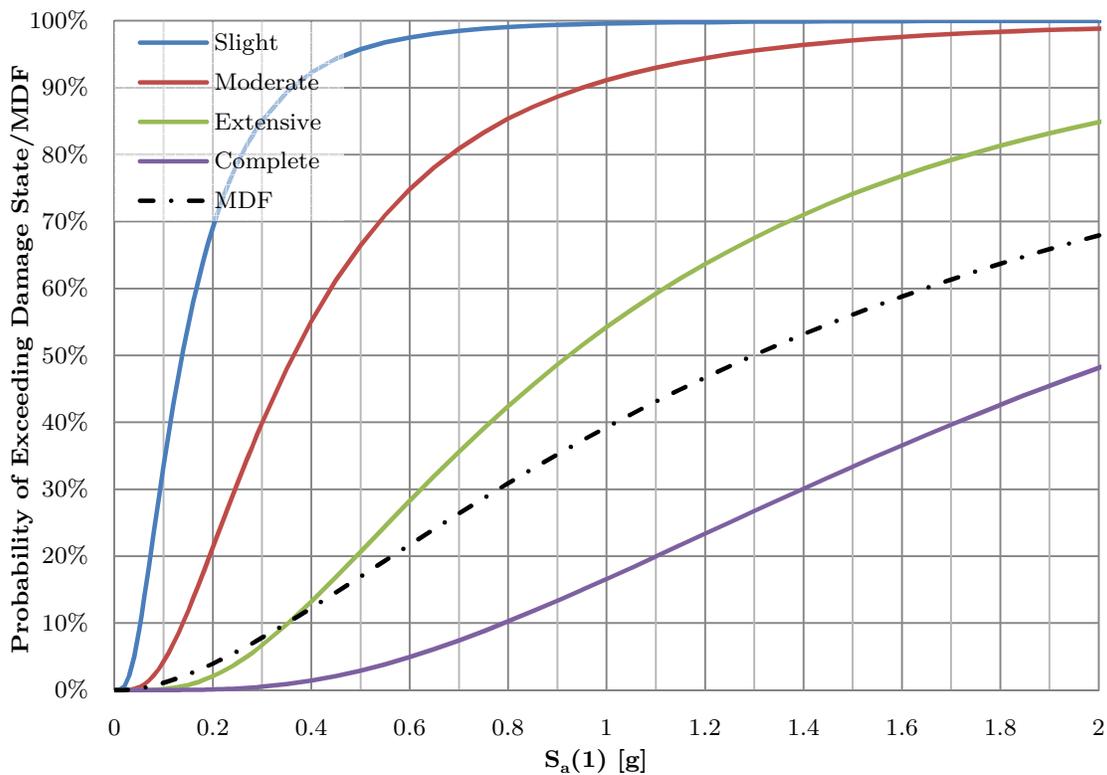


Figure 4.44 – HAZUS Struct. Fragility for Unretrofitted Type D/F Victoria

For example, at $S_a(1)=0.9g$, about 50% of the Canterbury buildings were in damage states of Heavy (Damage Ratio=45%) or greater (see Appendix B). Our HAZUS fragilities were proportioned to have approximately 50% of buildings in damage states of Extensive (Damage Ratio=50%) or greater – all while achieving the closest possible match between the MDF vs. $S_a(1)$ relationships based on the observed damage statistics (i.e. Figures 4.41 to 4.43) and those resulting from the combination of the damage state fragilities and their associated damage factors. Notice that the MDF curve in the above figure (i.e. the black dashed line) is nearly identical to the Unretrofitted, best estimate curve (i.e. the blue line) of Figure 4.41. The damage factors are specified in HAZUS and represent the loss as a fraction of the structural value of the subject building. They are as follows:

- None: 0%
- Slight: 2%
- Moderate: 10%
- Extensive: 50%
- Complete: 100%

To understand the relative performance improvements, it is perhaps more instructive to compare the individual damage states for various degrees of retrofitting. Figure 4.45 shows the various fragility curves for each damage state for a given retrofit level. As expected, parapet bracing and partial retrofits achieve nearly the same structural damage reduction as full retrofits at low levels of shaking, but with increased shaking intensity they converge to the unretrofitted case.

Note that the fragility curves were developed in terms of lognormal distributions so as to be compatible with the overall HAZUS framework in future loss estimates (although it would be necessary to convert back to terms of spectral displacement). While the preference would obviously be to produce fragilities based entirely on observed data, the results provided are thought to be a significant improvement over the existing data in HAZUS. Of course, care and judgment should be exercised before making use of these results in other studies because they could be at odds with the remaining structure type fragilities of HAZUS in a relative sense (i.e. a URM should be worse than a modern building). Results for the remaining building categories are provided in Appendix C.

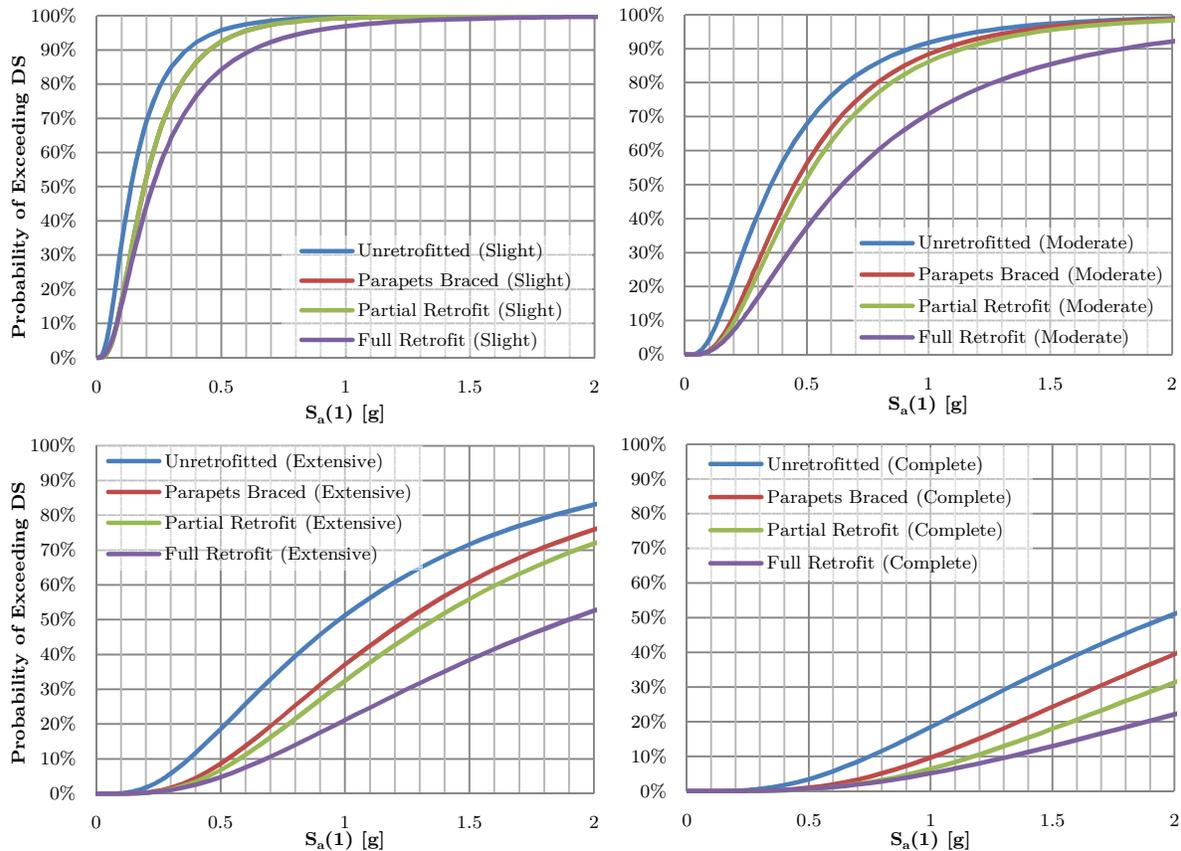


Figure 4.45 – Comparison of Damage State Fragilities by Retrofit Type

4.10 Summary

The purpose of this chapter was to establish motion-damage relationships for URM buildings based on observed damage statistics. To this end, the following was undertaken and was presented in this chapter:

Damage assessment methods for URM buildings and sources of damage information were reviewed:

- The ATC-13 damage scale was the primary scale used in collecting the damage data (which was performed by others)
- Other scales included were those from Wailes and Horner (1933) and ATC-20 (1989) (or highly similar variants)

Various ground motion intensity measurements and their appropriateness for use in developing motion-damage relationships was discussed:

- It was noted that because the purpose of this study was to produce loss estimates, it was necessary to make a compromise between more advanced IMs (eg. cumulative absolute velocity) and simpler parameters (eg. PGV, S_a)
- More advanced IMs likely would be better predictors of damage. Simpler IMs (eg. PGV, S_a) although likely not as accurate predictors of damage could be more readily related to existing information on seismic hazard and local (soils) effects
- Ultimately, $S_a(1)$ was selected as the IM for use in this study

Methods of estimating ground motion intensity at a site were reviewed and discussed:

- It was found that the weighted interpolation method as developed by Rutherford and Chekene (1990) was sufficiently accurate for the purposes of this study

The process of developing damage probability matrices (DPMs) and fragility curves from observed damage statistics was discussed.

URM damage statistics collected by others were obtained by the author and reviewed:

- Damage statistics from the 1989 Loma Prieta and 1994 Northridge earthquakes were kindly provided by Mr. Bret Lizundia of Rutherford and Chekene Engineers
- Damage statistics from the 2010/2011 Canterbury earthquake sequence were kindly provided by Dr. Jason Ingham of the University of Auckland
- Results from the Whittier earthquake were obtained from Wiggins et al. (1994)

Using the existing raw damage data, new motion-damage relationships were developed:

- Relationships for structural damage vs. $S_a(1)$ were developed for various URM strengthening levels for each of the data sets
- The relationships were defined in terms of HAZUS fragilities, since this is the current standard in loss estimates for North America
- As expected, the results indicated that limited retrofits (eg. parapet bracing, tension ties) achieve nearly the same results as full retrofit for low levels of shaking; however, the improvements disappear at high levels of shaking
- The results from the various data sets were compared and it was found that the Canterbury buildings suffered greater damage; however, the variation decreased with increased strengthening, as would be expected

Relationships for structural damage vs. $S_a(1)$ were defined for Victoria:

- A qualitative comparison was made between URM buildings for Victoria and Canterbury/California

- Low, high, and best estimates were defined using weighted average of the Canterbury and California databases
- The best estimate was ultimately based on 67%/33% weights for Canterbury and California, respectively; this was partially because the California databases included just low to moderate levels of shaking and it seemed imprudent to place more confidence in extrapolations than actual damage data

4.11 Conclusions

4.11.1 General Conclusions

The results showed that there can be significant differences in motion-damage relationships for similar types of buildings (i.e. URM) in different regions, which is thought to be due to both differences in construction and differences in the seismic demands that are not accounted for through our simple intensity measure (which is the 5% damped spectral acceleration at a period of 1 second). Examples include duration, directivity and the possibility for multiple earthquakes in a sequence.

The results showed that, as expected, limited strengthening measures such as parapet bracing and tension ties provide levels of damage reduction at low intensity of shaking that are similar to that achieved by full retrofits. However, the improvements disappear at higher levels of shaking.

Some differences were also noted between the observed results and the relationships from published sources such as ATC-13 (1985) and HAZUS (FEMA 2012), which all generally overestimated damage to some degree. The results presented herein are thought to be an improvement over the existing results, especially since no results for braced-parapet buildings were available.

4.11.2 Conclusions for Victoria

The most important conclusion for Victoria is likely that parapet-bracing can offer significant overall damage reduction for low-intensity shaking. The performance can be similar to more comprehensive retrofits at a fraction of the cost and this “low-hanging fruit” has not yet been pursued in Victoria.

Of course, it is also important to recall that the effects of parapet bracing disappear at higher levels of shaking, which could certainly occur.

4.11.3 Future Research Opportunities

Damageability effects of various building characteristics (eg. number of stories) have shown to be inconsistent to a certain degree in the literature. This is likely due to limitations in the data (both quality and quantity) as well as differences in the level of shaking. Much the same as the benefits of parapet upgrading disappear with increased shaking, it is postulated that differences in vulnerability due to slightly different building forms also likely disappear with increased shaking. This was accounted for conceptually in this study, but a more rigorous investigation is needed.

The selection of an intensity measure and interpolating the demands at each given site remains a challenge. The goal of this study was to prepare loss estimates using observed damage statistics – the goal was not to necessarily select the best possible IM or develop the best possible motion-damage relationship. However, improvements here would obviously lead to improved loss estimates (among many other improvements).

Chapter 5

Cost-Benefit Analysis for URM Seismic Rehabilitation

5.1 Purpose and Scope

In Chapter 3 it was seen that several communities had made the decision to implement URM seismic risk mitigation programs, which in many cases included some form of mandatory strengthening. However, these decisions were often founded on an emotional and/or political response to past earthquake losses. The purpose of this chapter is to provide an alternative, more rational motivation for URM seismic strengthening and thus avoid needless additional loss of life and property damage. To that end, cost-benefit analyses were undertaken and are presented herein. The analyses made use of the motion-structural damage relationships in Chapter 4, supplemented with additional relationships from other sources and cost data derived herein. The results are specific to Victoria, but the methodology can readily be extended to other communities.

5.2 Background

Quantification of the costs and benefits of URM seismic rehabilitation is a topic that has received some attention in the past, particularly in the United States during the 1980's and 1990's when many jurisdictions in California were in the midst of implementing URM retrofit ordinances. Example studies include those by Rutherford and Chekene (1990) and Recht Hausrath & Associates (1990, 1993). More recently, the City of Seattle has embarked on the process of implementing a URM seismic retrofit ordinance and commissioned a cost-benefit study (Gibson Economics 2014). While these studies illustrate the precedence for completing such an exercise, it was deemed worthwhile to perform a cost-benefit analysis in this study for the following reasons:

- No such study has been performed for a western Canadian city (and at the outset of this study, none in the pacific northwest)
- None of these studies were based on observed damage statistics – note that the Seattle study was based entirely on HAZUS results
- None of the studies included a “Parapets Braced” type of retrofit alternative, which was of particular interest in the current study

In this chapter, a summary is first provided of the costs and benefits typically considered as well as which stakeholders bear the costs and accrue the benefits. Subsequently, several relevant studies on URM cost-benefit analysis are summarized, the loss estimate and cost-benefit analysis methodologies used for this study are defined, and the results of a cost-benefit analysis for URM buildings in Victoria, BC are presented. Finally, a sensitivity analysis is presented and some of the limitations of cost-benefit analysis and decision-making based on expected cost are discussed.

5.3 Types of Costs and Benefits to be Considered

Table 5.1 summarizes the typical costs and benefits of a seismic upgrade. The primary cost in the analysis is the cost of retrofitting, while the benefits are essentially due to reduced expected losses in the form of damage, casualties, and downtime. There are a variety of stakeholders affected, including building owners, tenants, and the general public. Note that impacts may be costs for some and benefits for others; the manner in which they are presented is arbitrary.

Table 5.1 – Costs & Benefits for URM Seismic Upgrades

Cost	Description	Borne by
Building Retrofit	<ul style="list-style-type: none"> ▪ Construction costs ▪ Consulting costs ▪ Permit costs 	<ul style="list-style-type: none"> ▪ Building owners ▪ Building tenants (increased rent) ▪ Taxpayers (eg. grants/loans)
Disruption	<ul style="list-style-type: none"> ▪ Loss of income due to closure ▪ Loss of income due to reduced rent ▪ Noise pollution 	<ul style="list-style-type: none"> ▪ Building tenants ▪ Building owners
Loss of Historic Fabric	<ul style="list-style-type: none"> ▪ Demolitions due to seismic damage ▪ Demolitions instead of retrofits 	<ul style="list-style-type: none"> ▪ General Public
Benefit	Description	Accrued by
Reduced Damage	<ul style="list-style-type: none"> ▪ Reduced “expected” damage to building ▪ Reduced “expected” damage to contents 	<ul style="list-style-type: none"> ▪ Building owners ▪ Buildings tenants ▪ Insurers
Reduced Casualties	<ul style="list-style-type: none"> ▪ Reduced deaths ▪ Reduced treatment of injuries ▪ Reduced pain and suffering 	<ul style="list-style-type: none"> ▪ Buildings tenants ▪ General public ▪ Insurers
Increased resilience	<ul style="list-style-type: none"> ▪ Reduced loss of sales during recovery ▪ Reduced loss of rent during recovery ▪ Reduced regional/national funding ▪ Reduced loss of industry (businesses may choose to rebuild elsewhere after earthquake) 	<ul style="list-style-type: none"> ▪ Buildings tenants ▪ Building owners ▪ General public

Other costs and benefits could be defined. For example, local construction companies would likely benefit due to the high demand. However, the aforementioned impacts are those most likely to influence decision-making and so we limit our treatment to these specific impacts.

The degree to which different stakeholders are affected by these impacts will vary depending on a number of factors, including the building's use. Note that throughout the cost-benefit analysis only buildings of commercial (retail trade, or "COM1" as defined in HAZUS) occupancies are considered, as this is the most representative of the subject buildings in Victoria. The choice of occupancy affects the replacement value of the building and the relative proportions of the costs (i.e. structural, non-structural, contents). The difference between commercial and residential occupancies is of interest, since rehabilitated buildings are often converted from commercial to residential. Note that this applies more so to "full" rehabilitations, while a change of occupancy is much less likely to accompany a partial rehabilitation.

An examination of the 2012 HAZUS technical manual (FEMA 2012) shows that the replacement value, including contents, of a residential (RES3) building is about 25% greater than that of a commercial (COM1) building. The difference is mainly due to increased drift-sensitive non-structural components (i.e. partition walls). The increased exposed assets would increase losses and, thus, curtail the benefits of a retrofit to some degree.

With regards to life-safety, a review of the literature (NRC 1993, Rutherford & Chekene 1990) suggests that the occupant exposure (i.e. the number of occupants times the duration of occupancy) for residential buildings is equal to or less than that for commercial buildings. As such, there is likely some reduction in loss associated with the reduced occupant exposure and, thus, increased life-safety benefits.

Overall, it was felt that accounting for a conversion of occupancy associated with full retrofits in our cost-benefit analysis was not warranted, due to the lack of reliable information on the parameters driving the changes such as fragility of drift-sensitive non-structural components (NSC's) in URM buildings and detailed occupancy data. This level of evaluation is one that should be undertaken on a building-by-building basis. Of course, another issue not accounted for in the cost-benefit analysis is that commercial-residential occupancy changes are typically performed at the prospect of generating additional revenue.

5.4 Literature Review for Cost-Benefit Analyses

The opening section mentioned a few of the most relevant cost-benefit studies that were encountered in the literature. In this section, a brief summary of their methodologies and results is provided. In particular, we will review the studies completed for San Francisco (Rutherford & Chekene 1990) and Seattle (Gibson Economics 2014).

5.4.1 San Francisco Study

Two studies into the costs and benefits of URM seismic rehabilitation were commissioned by the City of San Francisco in 1990, when it was in the midst of implementing its URM retrofit ordinance. The work by Rutherford and Chekene (1990) characterized various retrofitting alternatives, including costs, disruption, and seismic performance. The work by Recht Hausrath and Associates (1990) addressed the socioeconomic impacts, including the expected number of building demolitions and effects on housing costs. Although neither of these reports explicitly provided a cost-benefit analysis, they provide essentially all of the components, with the exception of the economic analysis parameters (eg. time value of money).

The study by Rutherford and Chekene was quite broad in that it defined 15 different prototype buildings (recall Figure 2.2) and four strengthening statuses. The four strengthening statuses were defined as:

- 1) **Status Quo:** Unstrengthened (although many buildings had received parapet strengthening)
- 2) **Strengthening Option #1:** Out-of-plane wall strengthening (including tension anchors, shear anchors, and strongbacks for slender walls); commonly referred to as “bolts-plus”
- 3) **Strengthening Option #2:** Retrofit to the 1991 Uniform Code for Building Conservation (ICBO 1991) – this is similar to the current Canadian practices
- 4) **Strengthening Option #3:** Retrofit to San Francisco Building Code, Section 104f – this is similar to current code requirements for new buildings of the day

The purpose of the highly detailed prototyping was to better characterize the costs of seismic upgrading. This was possible and reasonable for the San Francisco buildings because Rutherford and Chekene had a complete database of the 2005 URM buildings in San Francisco and no dominant occupancy was observed. No such database is available for the Victoria buildings. However, a small sampling of buildings in a targeted area was completed as part of this thesis (see Section 7.2.2). Comparing the occupancies of this survey to the San Francisco database (Figure 5.1) shows that commercial buildings are

more predominant in Victoria. It should be noted that the Victoria statistics are based on floor area, while the San Francisco results are based on the primary occupancy for each building. The Victoria statistics may actually overestimate the prevalence of residential occupancies, since upper stories of buildings were assumed to be residential if no signage indicating otherwise was available at the ground level. These results reinforce the decision for selecting commercial occupancy as the only type for our analysis.

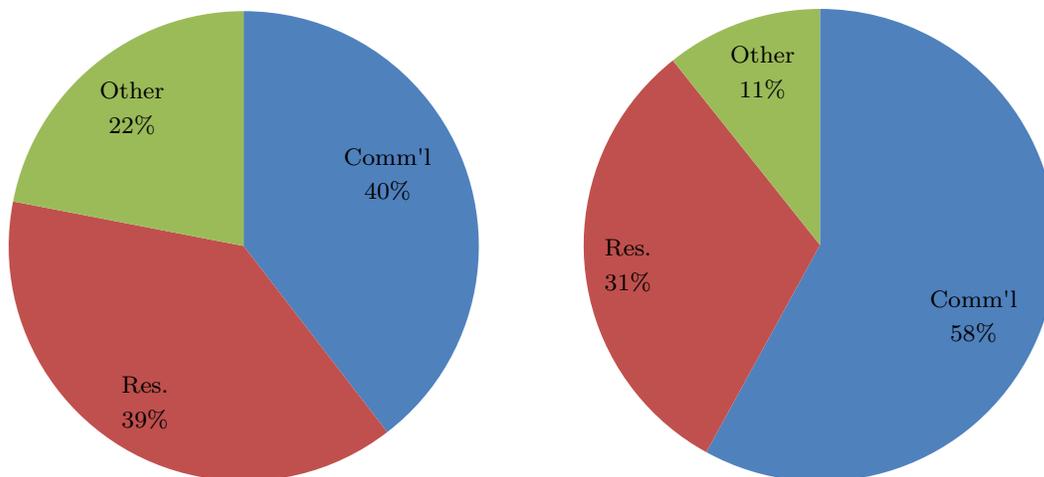


Figure 5.1 – Occupancy for San Francisco (left) vs. Victoria (right) URMs

The damage relationships were somewhat less refined: rather than defining 15 different relationships (for each strengthening status), the prototypes were grouped in terms of similar damageability based on engineering judgement and motion-damage curves were assigned to each group. Since retrofits were thought to homogenize the risk, fewer groups were developed for increased levels of retrofitting. Twelve damage curves (MDF vs. MMI) were developed in total, ranging between the ATC-13 values for URM (presumably low-rise) to Reinforced Masonry. Figure 5.2 provides a plot of the damage curves, including the ATC-13 curves. The nomenclature for the curves is as follows: the first character represents the strengthening status and the second character represents the enumeration of damage curve within a given strengthening status (eg. “1-2” represents the second damage curve for strengthening option #1). The higher enumeration indicates a higher vulnerability within that strengthening status (eg. 1-2 is more vulnerable than 1-1). Note that there is some overlap in vulnerability between the strengthening levels (eg. 1-2 is slightly more vulnerable than U-1).

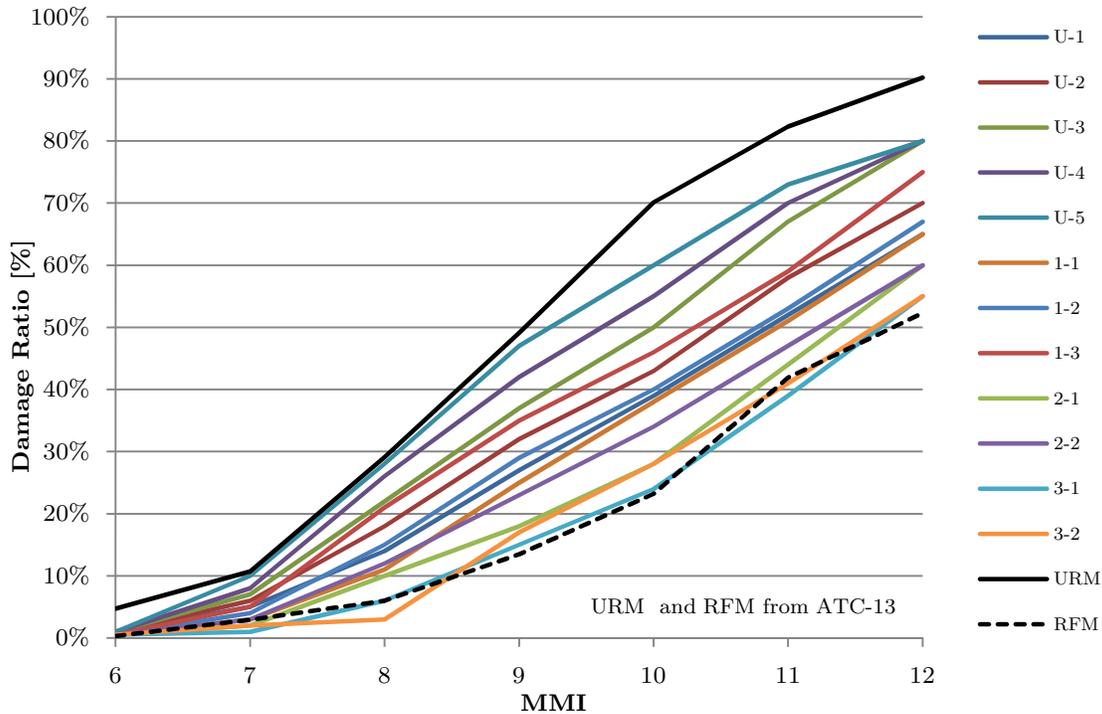


Figure 5.2 – Motion-Damage Relationships Used in San Francisco Study
(After Rutherford and Chekene 1990)

It is not explicitly stated, but these curves are assumed to represent damage costs as a fraction of the structural plus non-structural values, since this is the case in ATC-13 (i.e. contents were not included). All curves fall below the ATC-13 URM curve, consistent with our previous conclusions (Section 4.8.6) that the ATC-13 values overpredicted damage. However, it is interesting to note that the reinforced masonry case was selected as a lower bound; given the observation that ATC-13 overestimated damage to URM, it would seem plausible that damage for reinforced masonry is also overestimated. It is unclear if there should in fact be any relationship between damage to reinforced masonry and retrofitted URM buildings, as the structural systems are typically very different, with the possible exception of hollow core retrofits of URM walls.

Note that the curves used in the San Francisco study imply a significant degree of reduction not only for structural damage, but non-structural as well: a damage reduction of about 35% (50% to 15%) likely represents a value that is greater than the entire structural value. Structural components typically represent 15-25% of the building value (Onur 2001, Thibert 2008). At the opposite end of the spectrum, HAZUS (FEMA 2012) suggests modest reductions in non-structural damage as a result of retrofitting, particularly for acceleration-sensitive components, as will be shown in Sections 5.5.3.2 to

5.5.3.4. The issue of non-structural loss mitigation from structural retrofits is an important question, because NSC’s can easily represent 75% of the building value. This issue will be addressed later in this chapter when the non-structural/contents damage curves are defined for this thesis.

Rutherford and Chekene (1990) also accounted for casualties and downtime. The casualty estimates were developed by including outdoor casualties and calibrating ATC-13 fatality rates to match historical Californian earthquakes. Table 5.2 shows the fatality rates used. The study also provided indoor and outdoor occupant densities to which the fatality rates were applied. Finally, hospitalized injuries were assumed to be four times the number of fatalities.

Table 5.2 – Fatality Rates Used in San Francisco Study
(From Rutherford and Chekene 1990)

Strengthening Status	Location	Mean Damage Factor					
		0%	5%	20%	45%	80%	100%
Unretrofitted	Building	0	0.000010	0.00035	0.0035	0.035	0.20
	Street	0	0.000200	0.00300	0.0700	0.120	0.15
Retrofit Alt. #1 Out-of-Plane	Building	0	0.000009	0.00033	0.0034	0.035	0.20
	Street	0	0.000018	0.00280	0.0650	0.120	0.15
Retrofit Alt. #2 UCBC	Building	0	0.000007	0.00023	0.0028	0.035	0.20
	Street	0	0.000014	0.00200	0.0550	0.120	0.15
Retrofit Alt. #3 SFBC 104(f)	Building	0	0.000008	0.00030	0.0032	0.035	0.20
	Street	0	0.000016	0.00270	0.0600	0.120	0.15

Note: trailing zeros have only been shown to permit easy comparison of the relative values

The fatality rates above are based on the overall MDF (i.e. structural plus non-structural damage). Conversely, HAZUS makes use of the structural-only damage states to estimate casualties. While it could be argued that structural-only damage is better correlated with fatalities, the fact that the above rates were calibrated to actual earthquakes lends a significant amount of merit.

Estimates of downtime were accounted for using the ATC-13 data, with the exception that a “critical loss level” was included; this represents the damage level above which a building is likely to be demolished because repair is not economically viable. The result is that buildings below the critical loss level will be repaired (with an associated downtime), while those above this point will be demolished. Rutherford and Chekene (1990) chose to set the critical loss level at 40% and 50% for unstrengthened and strengthened buildings, respectively. Some studies (EERI 1989) have suggested critical

loss ratios of up to 65% - lower values were used by Rutherford and Chekene (1990) because URM buildings have a variety of other deficiencies aside from seismic vulnerability that may also require remediation and may be more likely to be not economically viable prior to the earthquake. In any case, the notion of a critical loss ratio appears intuitive and realistic and this will be implemented in the study at hand.

The study also quantified costs for the various retrofit options. The cost analysis was quite detailed in that square-foot costs for each prototype were generated based on data from the complete database of URM buildings in San Francisco and unit cost estimates submitted by a cost consultant. For example, Rutherford and Chekene had access to the height-to-thickness ratios of walls for each building, analyzed each prototype to see what fraction would require out-of-plane retrofits (eg. strongbacks) and included the cost of such work on a probabilistic basis for each prototype. Cost adjustments for work in occupied space, historically sensitive buildings, and work concurrent with architectural renovations were also provided. Table 5.3 shows the basic square-foot cost estimates developed by Rutherford and Chekene; note that the costs have been adjusted to 2014 Canadian dollars using RSMMeans' historical cost index (Reed Construction Data 2012) for comparison with the author's results later in this chapter.

Table 5.3 – Square-Foot Costs for Retrofits in San Francisco Study
Adjusted to 2014 Canadian Dollars
 (Modified From: Rutherford and Chekene 1990)

Prototype (see Figure 2.2)	Retrofit Alt. #1: Out-of-Plane		Retrofit Alt. #2: UCBC		Retrofit Alt. #3: SFBC 104(f)	
	Seismic	Arch.	Seismic	Arch.	Seismic	Arch.
A	18.88	1.38	20.51	1.54	27.30	1.83
B	10.95	0.56	15.44	0.76	18.18	0.97
C	9.06	1.32	14.64	2.20	23.12	5.13
D	10.95	0.74	14.99	1.38	26.65	3.56
E	18.59	0.37	21.50	0.95	29.34	2.08
F	8.46	0.21	14.13	1.46	17.13	2.39
G	23.37	1.98	25.99	2.14	34.35	3.62
H	10.65	0.78	15.71	1.24	19.75	3.27
I	16.99	1.65	28.99	2.24	40.89	4.69
J	8.65	0.80	16.14	1.63	25.74	4.02
K	20.53	3.77	22.55	4.12	32.02	6.18
L	11.88	2.20	15.36	2.68	20.92	4.84
M	11.84	1.65	29.63	2.97	33.54	5.19
N	7.68	1.03	18.49	1.44	29.10	4.88
O	14.50	2.00	20.57	2.10	27.84	4.82

As can be seen, the costs were broken into “seismic” and “architectural” costs: the seismic costs represent the work necessary to complete the retrofit (eg. including removal of finishes) while the architectural costs represent the work necessary to return the space to its original condition (eg. to re-instate or replace finishes that were removed). The architectural costs do not include extensive renovation works, which are frequently combined with comprehensive retrofit works.

Notice that the jumps in cost for increased strengthening vary by prototype. This is largely dependent on which elements would need work for a particular retrofit option. Prototype A, for example, has a relatively small jump between options #1 and #2; this is because the UCBC retrofit standard (i.e. Option #2) allows 1-storey buildings to have seismic resisting elements on only three sides, despite having a flexible roof diaphragm – as a result, new steel frames or concrete shearwalls would likely not be required. Obviously, the level of architectural finishes also affects the cost. The most relevant prototypes for our study are likely prototypes ‘G’ and ‘H’ (two/three storey office and commercial buildings, small and large area, respectively).

Again, such a database was not available for Victoria buildings and so this level of detail was not possible. Rather, we will focus on the “typical” commercial-occupancy type of building. In our study, we will develop unit cost estimates for certain selected retrofit works similar to those defined by Rutherford and Chekene.

Unfortunately, Rutherford and Chekene did not provide cost-benefit results for the various retrofit alternatives. However, the aforementioned parameters represent essentially all those required, except economic ones such as the time value of money.

5.4.2 Seattle Study

During the course of the writing of this thesis, the City of Seattle commissioned and released a cost-benefit analysis on URM seismic rehabilitation (Gibson Economics 2014). Below are some key points summarizing the analyses and the results. A discussion of the results and the merits and flaws of the study is provided below.

Results:

- The study’s best estimate of the cost-benefit ratio for retrofitting to bolts-plus was 7.6%, suggesting that the costs of retrofitting greatly outweigh the benefits;
- About 45% of the benefits were ascribed to economic savings (reduced damage and downtime) and 55% was ascribed to casualty savings

- A sensitivity analysis, including many parameters, suggested that the benefit-cost ratio could vary from 1.7% (worse case) to 66.8% (best case)

Analyses:

- The report considered only two levels of strengthening: “bolts plus” (most similar to San Francisco Option #1) and “reinforced” (most similar to San Francisco Option #2)
- The report contained an explicit cost-benefit analysis for each retrofit alternative
- The loss estimates were based on HAZUS data (eg. fragility curves, replacement values breakdown between structural and non-structural components)
- The study used three representative earthquakes and their annual probabilities to “approximate” the hazard curve
- Retrofit costs were based on discussion with local engineers and building owners
- Building values were specified based on local assessed values and assumptions were made regarding the contents values
- The number of stories of a building and the soils were accounted for through modified fragility curves

5.4.2.1 Merits

The greatest general merits of the study are that the results are current and that the economic analysis appears reasonably rigorous in that an extensive sensitivity analysis was performed. With regards to our study, it is particularly relevant because Seattle is likely the best proxy city for Victoria, due its relatively similar economic, social, and seismic environments.

5.4.2.2 Flaws

Despite the merits discussed above, there are also – in the author’s opinion – a number of flaws (i.e. opportunities for improvement in the current study).

The foremost flaw is likely the lack of technical rigor in defining the performance of the buildings. Although HAZUS is a very powerful and detailed tool, it appears that the ‘unretrofitted’ and ‘fully retrofitted’ strengthening statuses were characterized by default data (although a sufficient amount of information was not presented in the report to confirm this). In Chapter 4, we saw that the HAZUS curves overestimated damage in comparison to the observed data. As HAZUS tends to overestimate structural damage in both cases (both retrofitted and unretrofitted), the consequences may or may not be significant.

Additionally, there appeared to be inconsistencies in damage trends for the acceleration-sensitive components and building contents – under some of the scenario earthquakes, the results indicated a slight *increase* in damage for the ‘bolts plus’ retrofit versus unretrofitted. In the author’s opinion, this is highly unlikely since a bolts-plus retrofit would not add sufficient stiffness so as to induce higher accelerations. In fact, one would expect less damage due to a lower probability of structural failure (since the NSC’s will certainly be demolished if collapse occurs).

Compounding the aforementioned issue is that the report states that the value of the building contents was assumed to be equal to 50% of the building value (i.e. the structure plus nonstructural components). However, the HAZUS technical manual commonly specifies a contents value of 50-100% of the *structure* value (FEMA 2012). A contents value of 45% of the *building* value would be considered an upper bound, appropriate for high-technology buildings, while a more common value for contents would be 10-20% of the building value (Thibert 2008). Figure 5.3 shows breakdowns of the building value similar to that indicated in the report and a more plausible breakdown (within the 10-20% range noted by Thibert). Since the contents are also acceleration sensitive, they will be subject to the same discrepancy noted above. It is also significant because HAZUS specifies that the maximum loss of contents is 50% (because contents may be salvaged). Again, the report did not provide sufficient information so as to confirm the exact effects, but it appears that a more accurate representation could be achieved in our study.

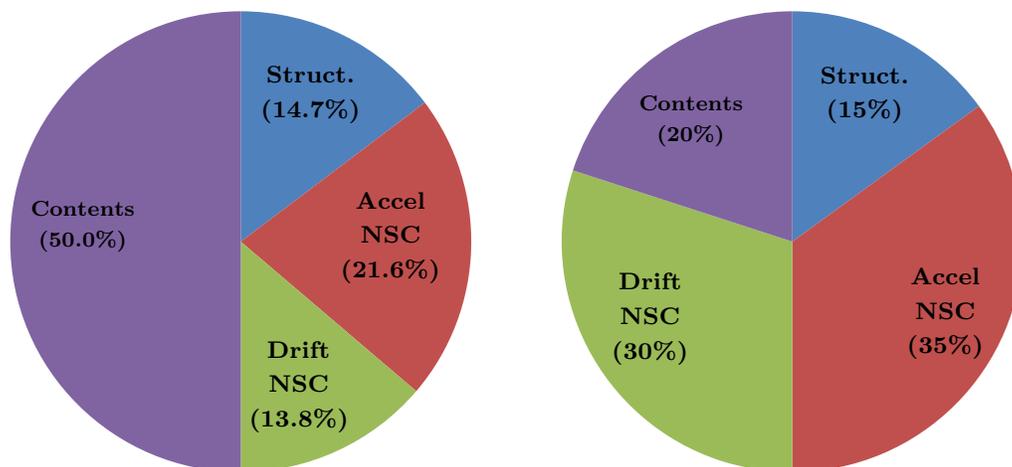


Figure 5.3 – Components for Seattle (left) and a More Common Assumption (right)

Another potential flaw in the Seattle study lies in the retrofit costs. The best estimate cost for a bolts-plus retrofit is stated to be \$40/sq.ft., with upper and lower bounds of \$20/sq.ft. and \$60/sq.ft., respectively. This was reportedly based on discussion with owners and engineers. A review of available data for Victoria (see Appendix A) shows that \$40/sq.ft. exceeds the mean cost for a *complete* retrofit to current code. Similarly, the data in Table 5.3 shows that the cost of a ‘bolts-plus’ type retrofit ranges between \$10/sq.ft. and \$25/sq.ft. While it may be pointed out that these costs do not include extensive renovations, this also appears to be the case in the Seattle study.

The treatment of the effect of soils is also worth discussing: in the study, the effect of soils is accounted for through adjusted fragility curves for “liquefiable” or “non-liquefiable” soils. The study does not describe how these adjusted curves were derived. While it is accepted that liquefiable soils may lead to increased damage, there are other types of soils (such as deep/soft clays) that also may be detrimental because of amplified ground motion. The study does not appear to address this issue. In our study, soils are accounted for using code-specified amplification factors, which effectively alter the probability of shaking at a site.

The study does not specify how non-structural damage is estimated. As non-structural components can make up the majority of the building value, accounting for loss reductions here is important.

One final item worth discussing is the possible larger-scale impacts resulting from widespread damage across a community. In the report, the indirect economic losses were assumed to be 15% of the direct physical damage to the buildings. However, as will be seen in Section 5.5.4, widespread damage can lead to economic impacts that are much more profound than can be represented with a simple ratio.

In summary, the report provides a valuable example of the cost-benefit analysis procedure, but it appears that a number of refinements can be made for our study.

5.4.3 Reinforced Concrete Upgrading Study

This study looked at hypothetical 1960’s vintage reinforced concrete (RC) buildings located in Los Angeles, as compared to various retrofit schemes and modern buildings. Expected annual losses were calculated using performance-based design assessment, applying incremental dynamic analysis over the range of possible shaking levels (Liel and Deierlein 2013). The losses considered included damage/repair to structural and non-

structural components and fatalities; downtime, contents damage, and non-fatal injuries were not included.

Building replacement values were derived from RSMeans Cost Data (Reed Construction Data 2012) and retrofit costs were based on FEMA 156 (FEMA 1994). The following economic parameters were specified for the cost-benefit analysis:

- Discount rate = 3% (for economic and life-safety benefits)
- Time horizon = 50 years
- Value of a Statistical Life⁸ = \$2million

The results showed that, when both economic and life-safety benefits were considered, many of the retrofit solutions had favorable cost-benefit ratios. However, retrofits could generally not be justified based on damage reduction alone. Of course, it is worth noting that the considered retrofits were in line with prevailing practices, which focus on life-safety rather than damage control. In general, life-safety benefits accounted for about 40-60% of the benefits.

5.4.4 New Zealand Studies

Having a relatively high level of public awareness on the topic, New Zealand has been grappling with seismic risk in its political arenas for over a decade, through its “earthquake-prone” building (EPB) policies. Two different cost-benefit studies specifically addressing the EPB policy were found in the literature: one from 2003 (Hopkins and Stuart 2003) and another from 2012 (Martin Jenkins Associates 2012).

5.4.4.1 2003 Study

Since the EPB policy applies equally to any construction type, the buildings are arranged by vintage rather than construction type. Four vintages are considered: pre-1935, 1935-1965, 1965-1976, and post 1976. The study considered the costs and benefits of three alternatives (NBS refers to “New Building Standard”):

- 1) status quo (no strengthening)
- 2) Strengthening to 34% NBS
- 3) Strengthening to 67% NBS

The 2003 study looked at 17 different locations around New Zealand of varying seismicity. The motion-damage relationships used were judgementally derived,

⁸ See Section 5.5.6.4 for discussion on the meaning of the "Value of a Statistical Life" and how it may be derived

somewhat similar to the provisions of ATC-13. The remainder of the process was similar to the analyses undertaken by the author in that annual probabilities were used to determine losses (economic and life-safety) and economic parameters were used to convert the losses to present value, for comparison with retrofitting costs. The resulting benefit/cost ratios varied greatly from location to location (due to seismicity) and building vintage (due to vulnerability). For example, strengthening pre-1935 buildings in Wellington to 34%NBS showed a benefit/cost ratio of 3.3, compared to just 0.4 for Christchurch or 0.03 for Auckland. Strengthening of more recent vintages showed less favourable results. Strengthening to 67% for Wellington showed more favorable results (B/C=5.2) for pre-1935 buildings.

5.4.4.2 2012 Study

After the 2010/2011 Canterbury earthquakes and the resulting relatively poor performance of buildings strengthened to 34%NBS (see Figure 3.2), interest in strengthening to 67%NBS rather than the currently legislated 34%NBS grew and the Ministry of Business, Innovation, and Employment commissioned a new cost/benefit study (Martin Jenkins Associates 2012).

The study examined the cost/benefit of increased retrofitting levels and shorter time frames (the current average deadline of the territorial authorities is 28 years), as follows:

- 1) Strengthening to 34%NBS within 15 years
- 2) Strengthening to 67%NBS within 15 years
- 3) Strengthening to 100%NBS within 15 years

Similar to the 2003 study, the cost/benefit analysis is performed for buildings based on vintage (the same categorization), rather than construction type. Retrofit costs were based on engineering estimates for various types of construction, grouped into the aforementioned vintages. Unfortunately, the available documentation does not indicate how the motion-damage relationships were derived. One interesting feature of the study is that it accounts for time-lag effect of the policy (i.e. not all buildings could be strengthened tomorrow). However, this seems to be an unnecessary complication, given the broad nature of the study. The results presented in the available documentation (which appears to be a summary report) provide cost/benefit data only for New Zealand as a whole. The B/C ratios ranged from about .01 to .02 for the increased strengthening options, and .026 for the current legislation. These results are highly unfavorable, although it must be recalled that B/C ratios varied greatly (by about two orders of

magnitude) from location to location in the 2003 study. Unfortunately, sufficient data was not provided in the available documentation to permit a more critical review.

5.5 Development of Loss Estimate Methodology

The benefits associated with URM seismic rehabilitation lie in the reduced “expected losses” due to earthquakes. This section reviews the losses that are to be considered and discusses the methodology used to estimate the losses. The material is presented in more detail than is needed to understand the subsequent cost-benefit analysis, but this was thought to be worthwhile since loss estimation involves a significant amount of judgement and future analysts will likely wish to make further refinements. Those interested strictly in the results may proceed to Section 5.6. In this study, we will consider the following losses:

- Repair/replacement costs (due to damage from ground shaking)
- Casualties
- Lost rental income (during post-earthquake downtime)

These are not the only possible losses, but have traditionally been the most commonly analyzed and are thought to be the greatest sources of loss. Other possible losses not considered here could include:

- Damage to buildings due to geological hazards (eg. landslides)
- Damage due to inundation (tsunami and seiche)
- Damage due to fire, caused by earthquakes
- Social losses (eg. displaced households, loss of historic fabric)

The current standard in loss estimating is the use of HAZUS (FEMA 2012), which is based primarily on analytical methods. As discussed in Chapter 2, URM buildings behave differently from most other buildings and – particularly in the case of unretrofitted buildings – are difficult to model analytically. Furthermore, it was illustrated in Chapter 4 that the existing motion-damage HAZUS relationships for URM structures were not consistent with the results of damage surveys. Therefore, the motion-damage relationships of Chapter 4 will be used in the estimates of structural damage. For non-structural components and contents, existing loss data in HAZUS will be employed, with some adjustments. An attempt will also be made to capture longer term economic losses due to larger-scale disasters based on recent experiences in Christchurch, New Zealand. Figure 5.4 provides a simplified flow chart showing the loss estimation process. More detailed flowcharts and discussions can be found in the literature (Rutherford & Chekene 1990, Thibert 2008, FEMA 2012).

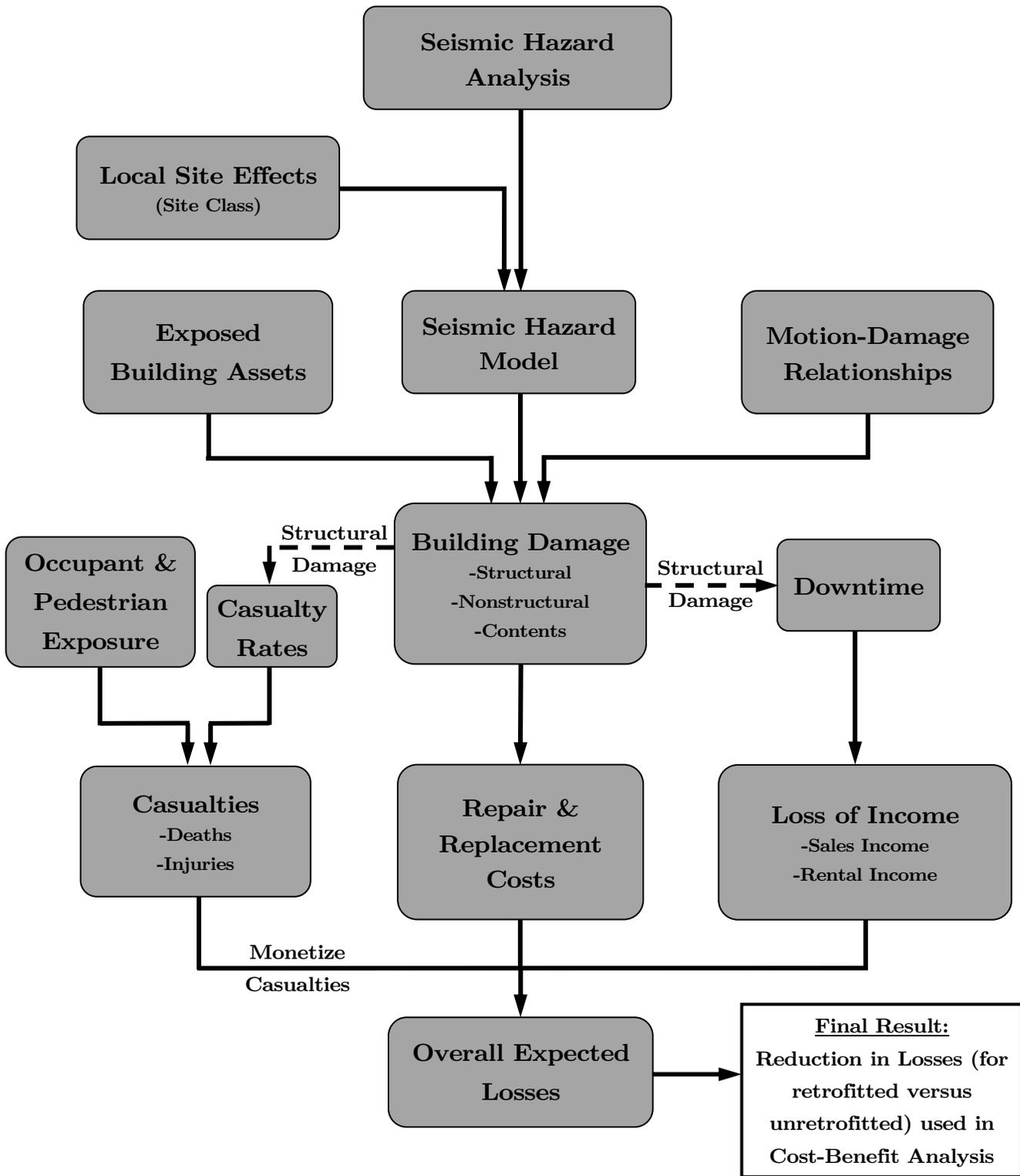


Figure 5.4 – Flow Chart of Loss Estimation Process used in this Study

5.5.1 Seismic Hazard Model

The seismic hazard model used in this study is similar to that underpinning the seismic hazard design values for the 2015 National Building Code of Canada (NBCC). This model was used in favour of the values behind the 2010 NBCC hazard values because it includes the hazard from the Cascadia subduction earthquake event on a probabilistic basis; this significantly increases the hazard for Victoria. Local site effects were accounted for by applying the NBCC-specified long period amplification factor (F_v) to the $S_a(1)$ values from the seismic hazard analysis. Appendix D contains the resulting hazard values for each site class.

5.5.2 Exposed Building Assets

Buildings are composed of many components, all of which are assets and are potentially subjected to damage in an earthquake. The vulnerability of different components will vary (eg. contents on store shelves may suffer damage at a much lower shaking than the structure itself) and therefore it is beneficial to separate components by vulnerability. In this study we will make use of the component classifications from HAZUS, as shown below:

- Structural components
- Drift sensitive nonstructural components (eg. partitions)
- Acceleration Sensitive Nonstructural components (eg. parapets)
- Building contents (eg. Furniture)

HAZUS also defines business inventory as a separate component, but here, it is included with the contents, since their fragility is identical and we are considering only a single occupancy type: commercial retail (or ‘COM1’ as defined by HAZUS). Figure 5.5 shows a breakdown of the building value by component that will be used in this study.

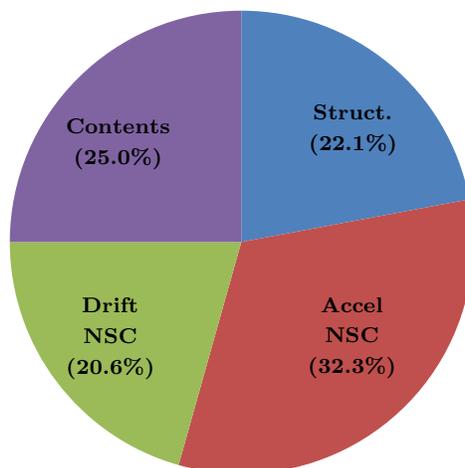


Figure 5.5 – Component Breakdown used in this Study

For a URM building, the notion of a replacement value is somewhat complicated by the fact that this type of construction is now prohibited by law in many places. Rutherford and Chekene (1990) suggested the value of a concrete shear wall building be used, as it was deemed to be the most likely replacement; this seems particularly true for southwestern BC where concrete is a favored type of construction. Thibert (2008) provides replacement values for BC buildings and so the corresponding (2014 CAD) value of \$260/sq.ft. was used.

5.5.3 Motion-Damage Relationships

Each of the aforementioned components is assigned its own motion-damage relationship that is characterized by 1) damage state fragilities and 2) loss values corresponding to each damage state. The sections that follow discuss the motion damage relationships.

5.5.3.1 Structural Components

As aforementioned, we will make use of the motion-damage relationship for structural components that we derived Chapter 4 (Section 4.9) based on observations from past earthquakes. There are actually eight relationships in total since we have four retrofit categories and vulnerability adjustments based on the typology for the unretrofitted and braced parapet buildings. Figure 5.6 shows the MDF (or Damage Ratio) vs. $S_a(1)$ relationships for each case.

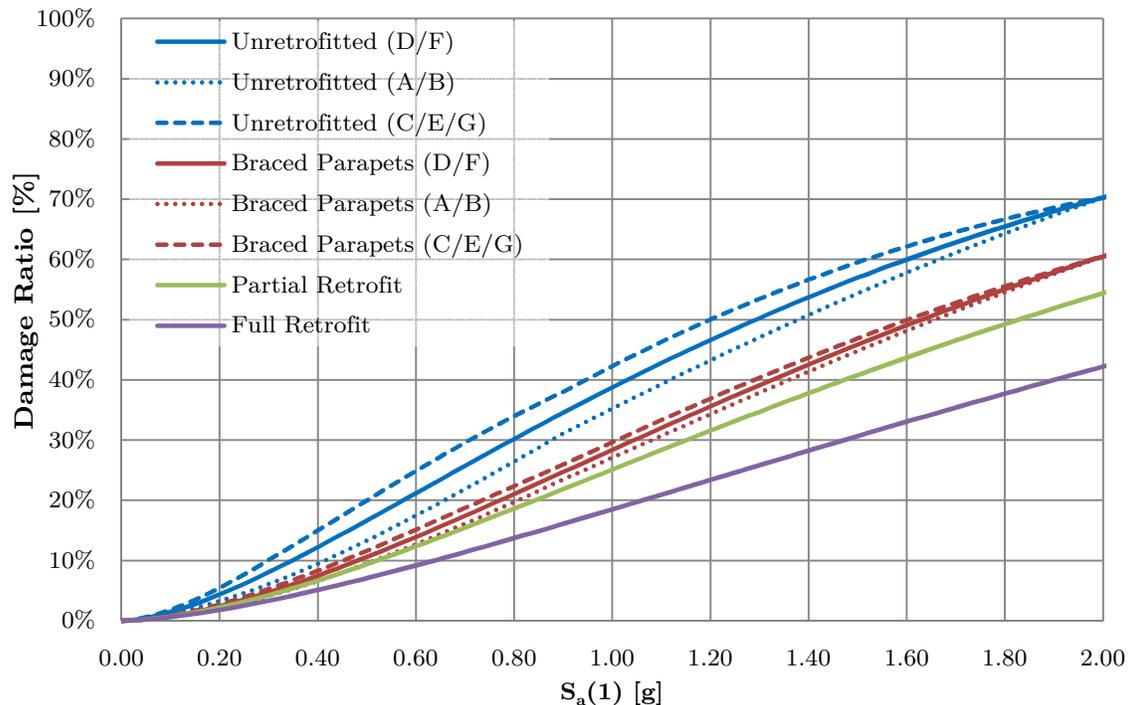


Figure 5.6 – MDF vs. $S_a(1)$ Relationships (Best Estimate)

Notice that the curves of a given retrofit status eventually converge, as a result of the modification factors from Table 4.22. This reflects the observation that the *relative* impact on damageability due to certain characteristics decreases with increasing damage – i.e. a one-storey building may fare significantly better than a 3-storey building under modest ground motions (such as those from Loma Prieta), but this becomes less important at more severe ground motions (such as those from Christchurch).

Finally, for the case of structural components, the damage states are associated with the following damage factors (as specified in HAZUS) which represent the monetary loss as a fraction of the *structural* value.

- None: 0%
- Slight: 2%
- Moderate: 10%
- Extensive: 50%
- Complete: 100%

Figures 5.7 and 5.8 provide the fragility curves for unretrofitted and fully retrofitted buildings, respectively. These were derived based on observed damage in past earthquakes, as discussed in Section 4.9.3. The remaining plots are contained in Appendix C.

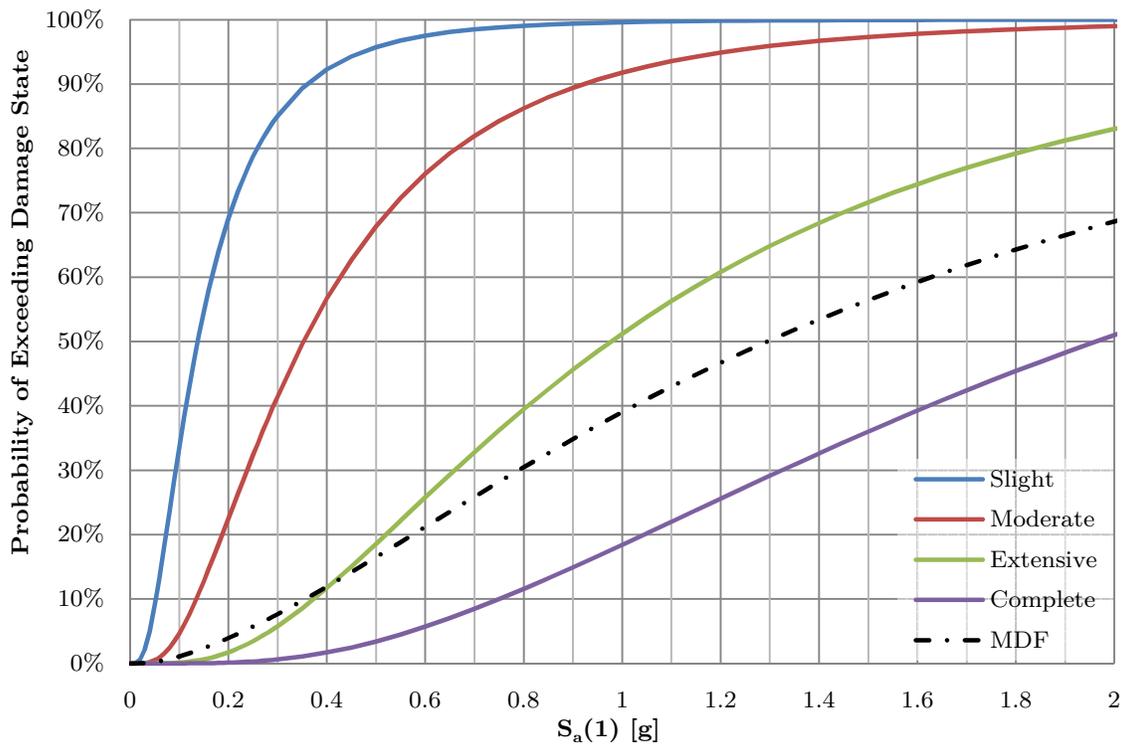


Figure 5.7 – Structural Fragility Curves for Unretrofitted Buildings

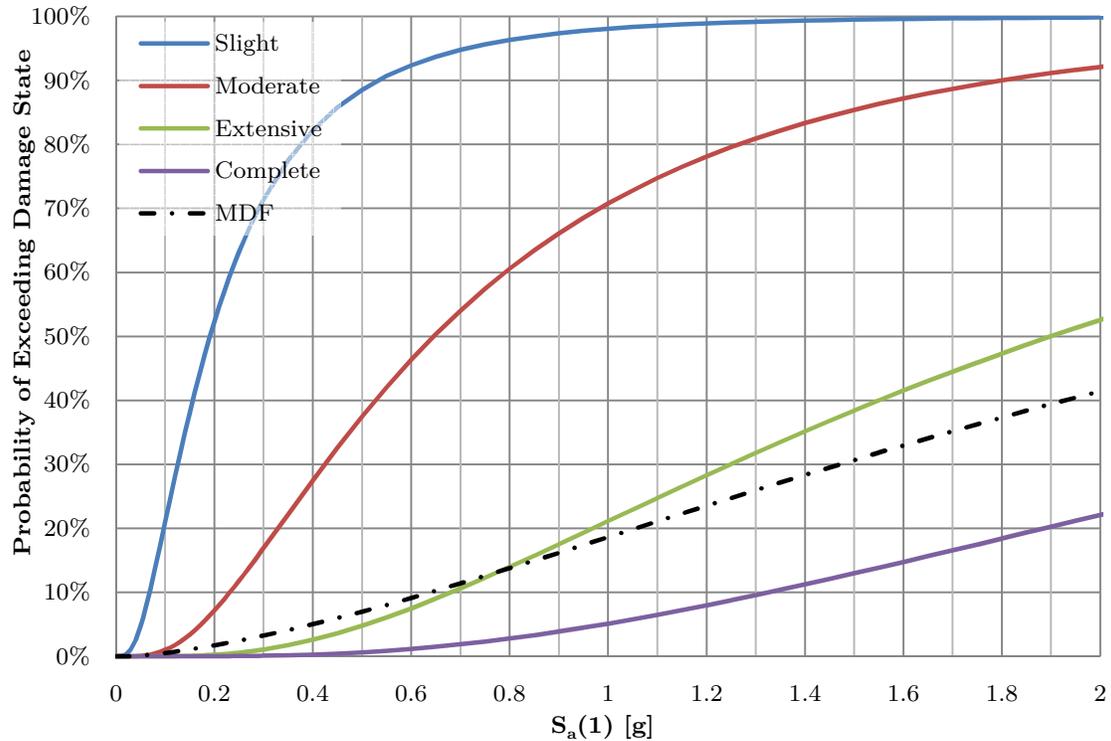


Figure 5.8 – Structural Fragility Curves for Fully Retrofitted Buildings

5.5.3.2 Drift-Sensitive Non-Structural Components

Such a level of refinement in the assessment of non-structural components was not possible, due to a comparative lack of observed damage data specifically within unreinforced masonry buildings. For the study at hand, we have made use of the existing data in HAZUS, with some adjustments as subsequently discussed. The damage states and loss values are the same as for the structural damage.

For unretrofitted buildings, we started with the “URML (Precode)” fragility data from HAZUS. For fully retrofitted URM buildings, HAZUS (FEMA 2012) recommends the use of the fragility data for essential facility reinforced masonry buildings with flexible diaphragms, “RM1L (moderate code),” and we have followed these recommendations. This seems reasonable as a retrofitted building is likely to have its drifts reduced by new vertical seismic resisting elements and/or diaphragm strengthening. Interestingly, there was relatively little difference between the corresponding essential and non-essential facility data from HAZUS. Figures 5.9 and 5.10 show the fragility curves for unretrofitted and retrofitted buildings, respectively.

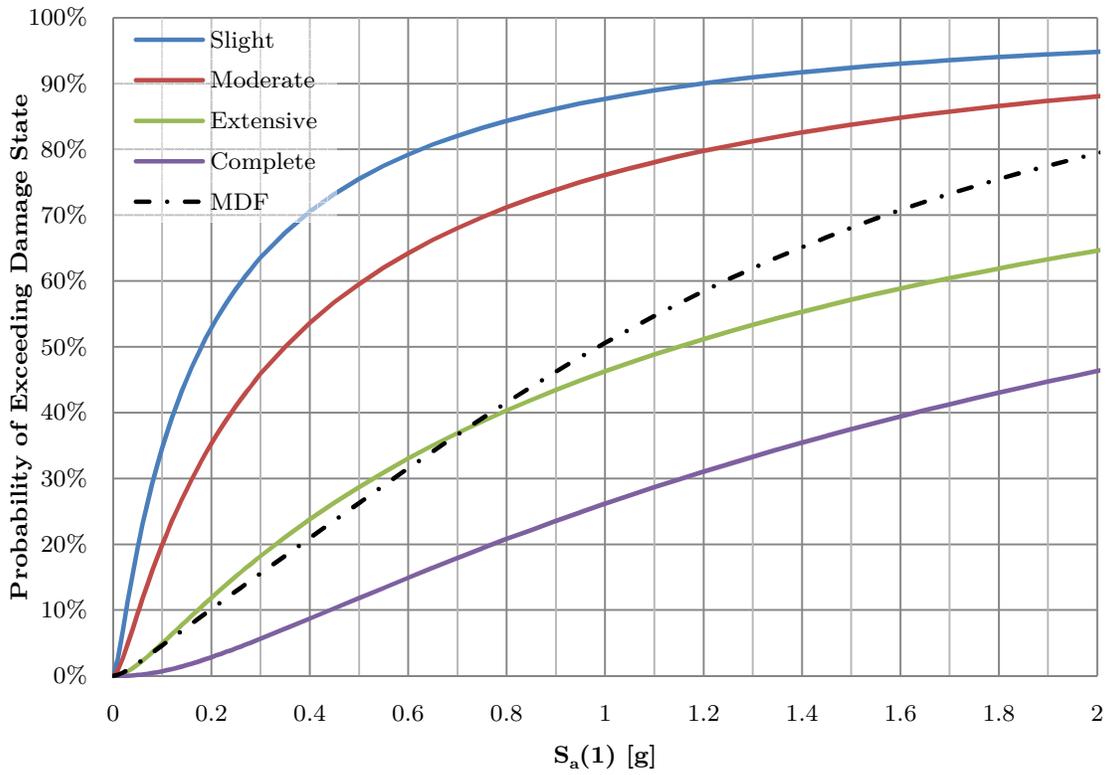


Figure 5.9 – Drift-Sensitive NSC Fragility Curves for Unretrofitted Buildings

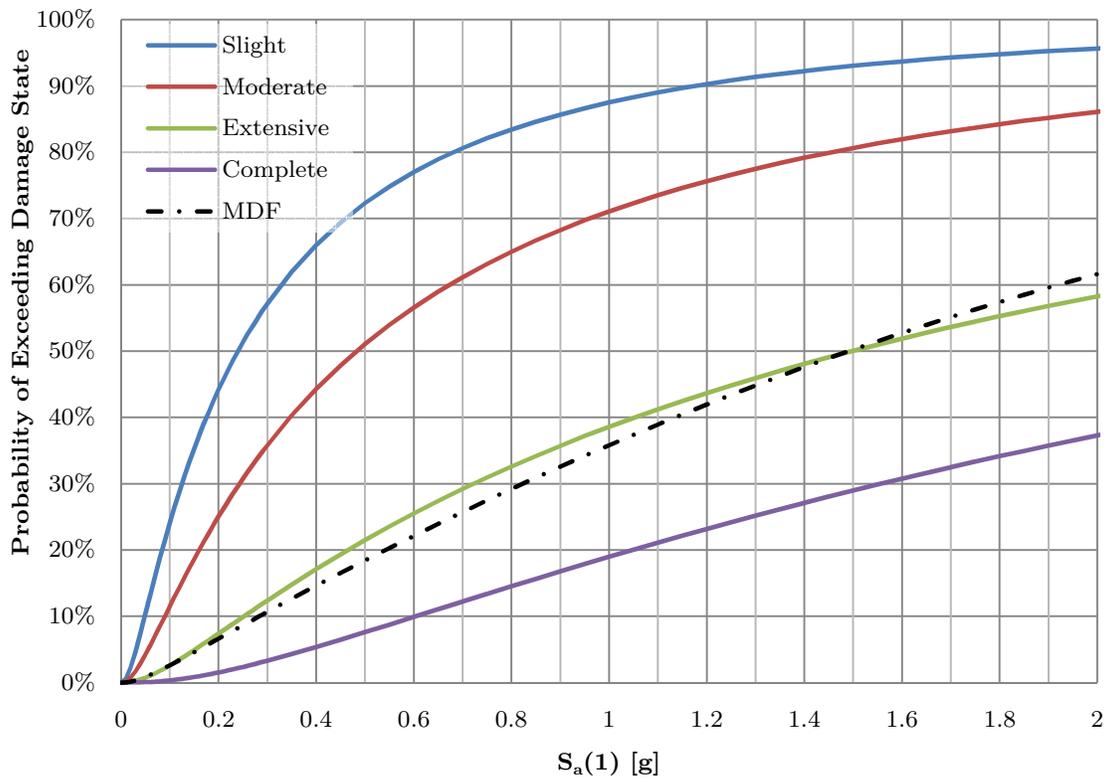


Figure 5.10 – Drift-Sensitive NSC Fragility Curves for Retrofitted Buildings

HAZUS does not provide recommendations for partially retrofitted buildings. However, we should expect some reduction in damage to drift-sensitive NSC's: cracking of plaster due to separation of walls/floors is a common type of damage and even the “braced parapet” or “partial retrofit” type strengthening measures would help prevent this. Ultimately, it was decided to use HAZUS' data for “RM1L (Low Code)” buildings; the resulting changes to the fragility curves were quite modest and may understate the improvements. These curves can be found in Appendix C.

It should be noted that HAZUS provides the curves in terms of spectral displacement at a period of 0.35seconds (the period assigned to URML). In order to use these curves, it was necessary to convert them to $S_a(1)$. This was achieved by first applying typical pseudospectral conversion relationships (Chopra 2012) and then scaling the resulting acceleration by the ratio of $S_a(.35sec)/S_a(1.0sec)$ for Victoria's 2475-year site class 'C' uniform hazard spectrum.

One shortcoming of the HAZUS data is that it does not account for the effects of collapse on non-structural damage (Farokhnia 2013, Thibert 2008). Thibert accounted for collapse effects by substituting the structural MDF for all three non-structural MDF's once it exceeded 60%. Farokhnia accounted for collapse effects by dividing the MDF into ‘collapse’ and ‘non-collapse’ portions, similar to equation 5-1 below (modified here to be consistent with our overall methodology).

$$\text{Total NSC Loss} = \%_{\text{collapsed}} * RV_{\text{NSC}} + (1 - \%_{\text{collapsed}}) * MDF_{\text{NSC}} * RV_{\text{NSC}} \quad (5-1)$$

Where:

$\%_{\text{collapsed}} \equiv$ the fraction of buildings expected to collapse

$RV_{\text{NSC}} \equiv$ the replacement value of the non-structural components

$MDF_{\text{NSC}} \equiv$ the mean damage factor of the non-structural components

This approach seems rational and avoids a sudden step in the MDF curve. For this study, we have used a similar approach. However, we have made a further modification in that we have used the “Complete” damage state as the dividing line. The reason for this is that many buildings that do not collapse as a result of the seismic damage will also likely be demolished because repair is not economically viable and it is assumed that the vast majority of drift-sensitive non-structural components (eg. partitions, windows) will not be salvaged – this adjustment was included in the foregoing figures. This same assumption was applied to the acceleration-sensitive NSC's and contents (although some salvage of contents is accounted for as discussed in Section 5.5.3.4).

5.5.3.3 Acceleration-Sensitive Non-Structural Components

For the acceleration-sensitive non-structural components, we have again made use of the existing data in HAZUS. The damage states are again the same, but the loss value for the ‘Extensive’ damage state is just 30%, as shown below.

- None: 0%
- Slight: 2%
- Moderate: 10%
- Extensive: 30%
- Complete: 100%

For unretrofitted buildings, we again assigned the “URML (Precode)” values. However, for partial retrofitted buildings, it was decided not to use the essential facility values: this is because the values for the acceleration sensitive NSC’s have been increased by 50%, representing a typical code importance factor, which is not representative for the majority of URM retrofits. Instead it was decided to use the “RM1L (High Code)” values, which are only 50% greater than the “precode” values. This was felt to be reasonable as seismic restraint (to current code) is typically included in comprehensive retrofits in Victoria. Figures 5.11 and 5.12 show the fragility curves for acceleration-sensitive components in unretrofitted and retrofitted buildings, respectively.

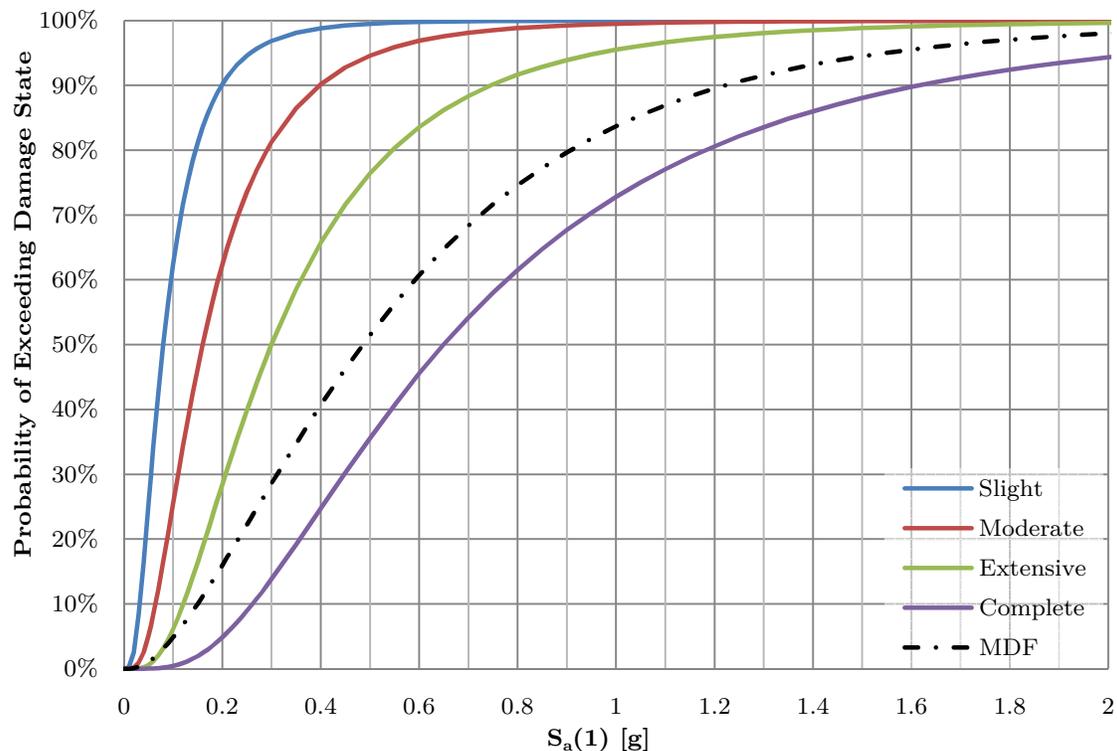


Figure 5.11 – Acceleration-Sensitive NSC Fragility for Unretrofitted Buildings

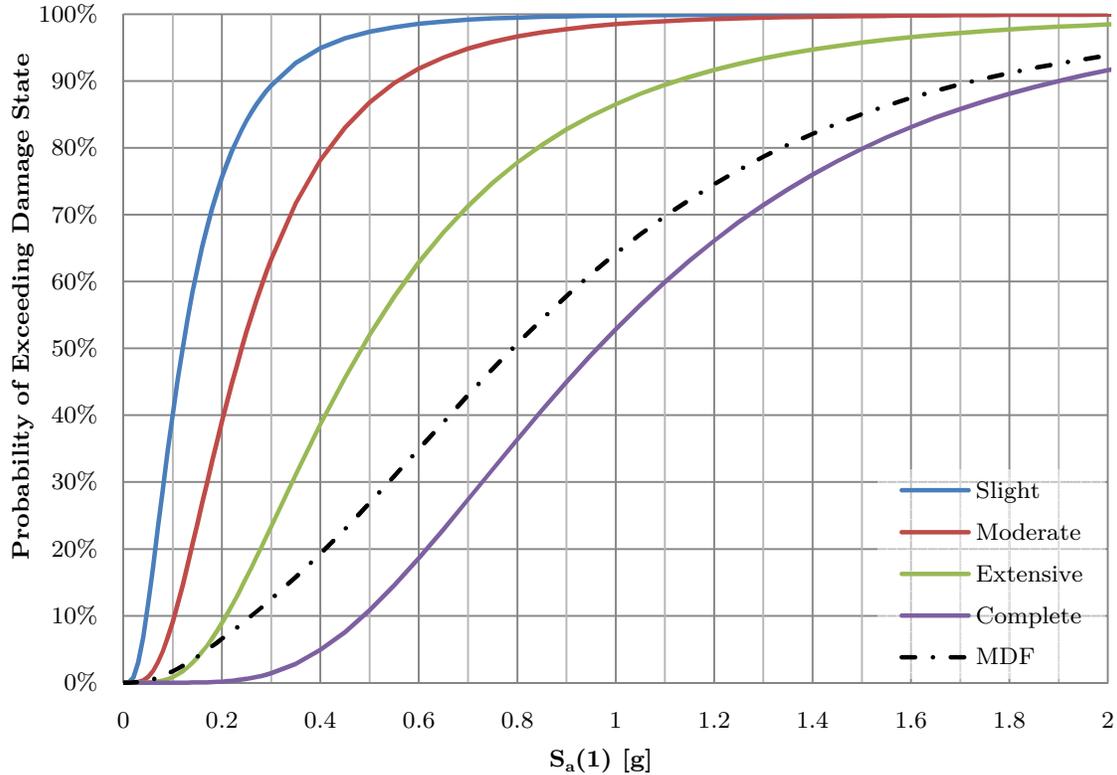


Figure 5.12 – Acceleration-Sensitive NSC Fragility for Retrofitted Buildings

Braced parapet buildings were assigned the same fragility as the unretrofitted buildings, since it was felt that few of the acceleration-sensitive NSC's would be impacted. Since parapet damage was included in the observed damage statistics, this was assumed to be fully accounted for by the structural fragilities.

For the partially retrofitted buildings, it was felt that some appreciable improvement would be achieved; it was decided to define the fragility data as the average of the “precode” and “moderate code” values for RM1L – this resulted in curves with about 33% of the improvement achieved in a full retrofit building, as discussed above.

The improvement due to reduced collapse/demolition rates is also accounted for in all cases, similar to that of the drift-sensitive components.

5.5.3.4 Building Contents

In HAZUS, building contents are assumed to be acceleration sensitive and so those damage state fragilities are assigned. However, the loss values associated with each damage state are reduced. This represents the fact that a portion of the contents are assumed to be salvaged. The loss values are shown below:

- None: 0%
- Slight: 1%
- Moderate: 15%
- Extensive: 25%
- Complete: 50%

The damage state fragilities used were the same as those for the acceleration sensitive NSC's, as discussed above and the collapse/demolition effects were again included. Figures 5.13 and 5.14 show the resulting fragility curves for building contents in unretrofitted and retrofitted buildings, respectively.

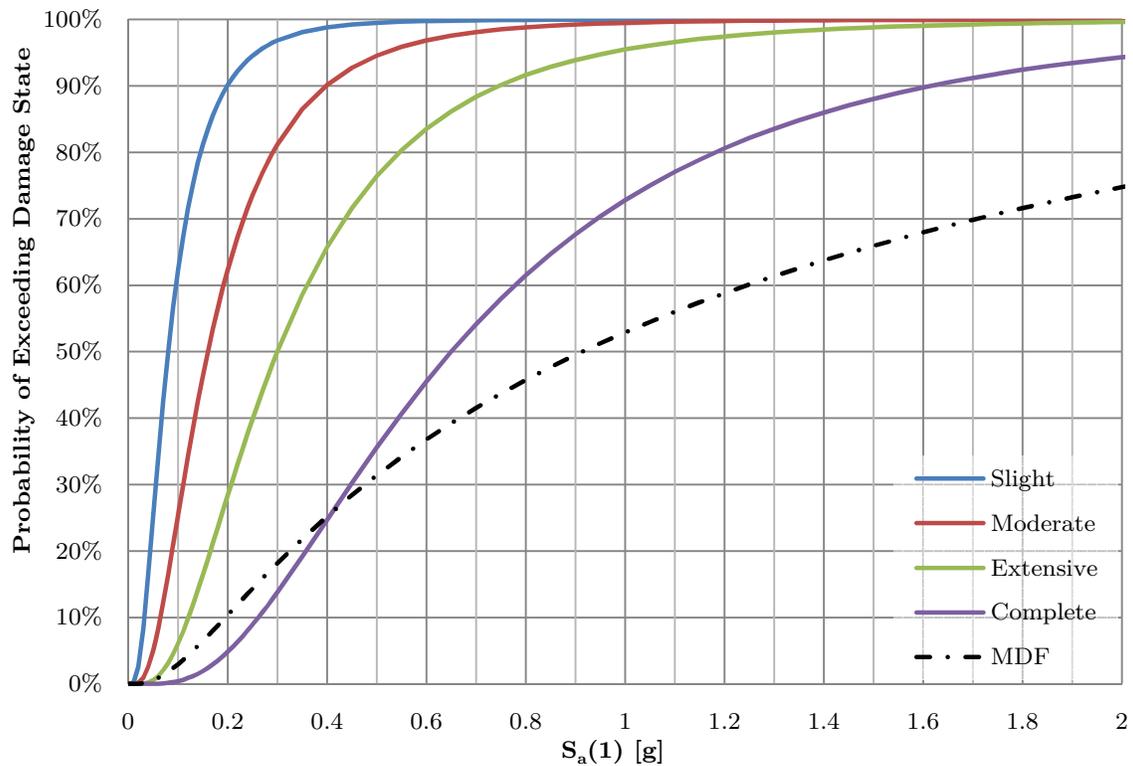


Figure 5.13 – Contents Fragility for Unretrofitted Buildings

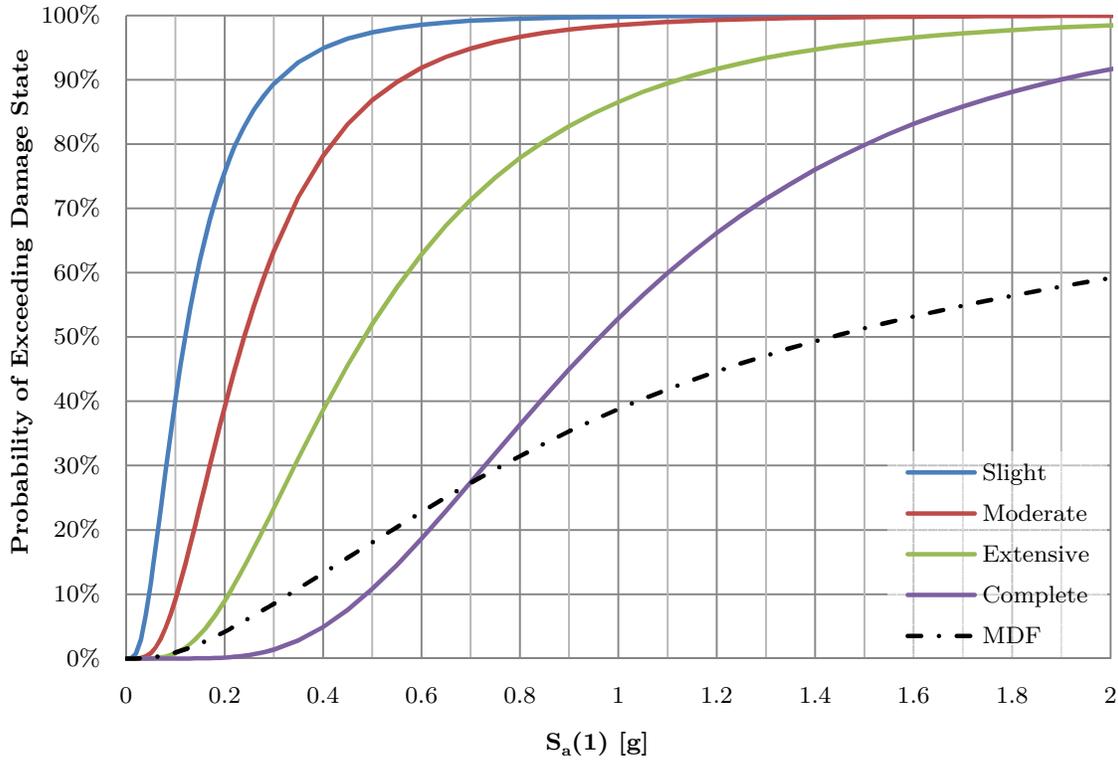


Figure 5.14 – Contents Fragility for Retrofitted Buildings

5.5.3.5 Overall Damage

Having developed the fragility curves for each component (structural, acceleration NSC, Drift NSC, contents), the overall motion-damage relationship for the subject building can be derived by taking the average of these four components, weighted by their fraction of the building replacement value as shown in Figure 5.5. The resulting overall MDF vs. $S_a(1)$ relationships are shown in Figure 5.15. These will be used to determine the losses due to repair/replacement costs in Section 5.6.

The trends of improvements in the overall damage are somewhat different from those for the structural damage (see Figure 4.45): the “Parapets Braced” and “Partial Retrofit” curves initially begin to approach the full retrofit curve and then subsequently re-converge with the unretrofitted case. This is primarily due to the collapse effects on the non-structural components, which are obviously most significant in the middle region. The initial differences are primarily due to reduced damage to the structure and acceleration-sensitive NSC’s. As will be seen in the subsequent loss estimates, the values beyond about 2g are of very little significance since the probability of occurrence is extremely small, but they were shown here so as to illustrate the overall trends.

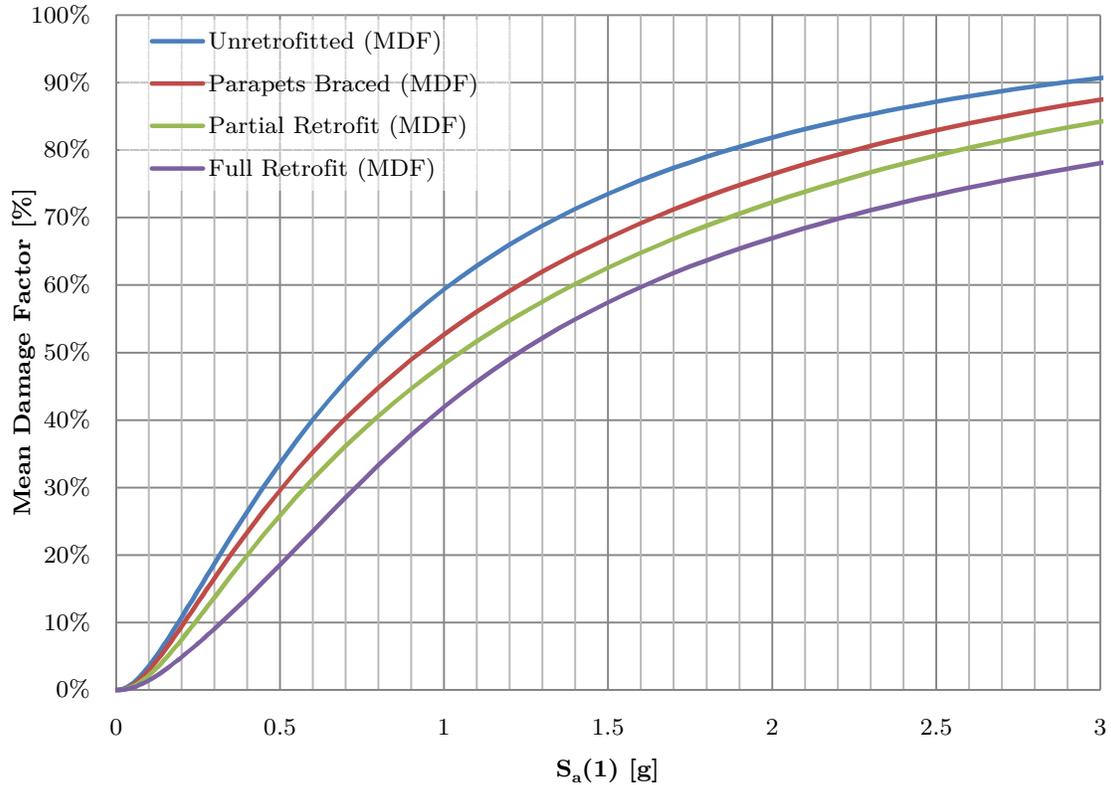


Figure 5.15 – Overall Motion-Damage Relationships used in This Study

In determining overall building damage costs, many studies (Rutherford & Chekene 1990, EERI 1994) have also incorporated the notion of an economic critical loss ratio (ECLR). This represents the damage value at which a building is likely to be replaced rather than repaired; the rationale is that unreinforced masonry buildings may be deficient in a variety of manner (eg. fire safety, access, energy efficiency) and at some point, these added improvements will make up for the difference in repair versus replacement costs. The Christchurch earthquake provides a real-world example: the average MDF based on the damage surveys was about 45% and widespread demolitions ensued (Moon, et al. 2014). Rutherford and Chekene (Rutherford & Chekene 1990) used an ECLR of 40% for unretrofitted buildings and 50% for retrofitted URM buildings. EERI (1994) recommended an ECLR of 65%, but this was a general value not specifically intended for URM buildings.

It is acknowledged that the damage to a building is but one of many factors that contribute to the decision to demolish a building. Marquis et al. (2015) provides a detailed treatment of factors leading to building demolitions. For the purposes of this study, however, we will simply use an ECLR of 50% for all buildings.

5.5.4 Downtime

Earthquake damage to buildings results in downtime for initial building assessments, cleanup, economic analysis, and – depending on the level of damage – repair or replacement. Both the building owner and tenants can incur losses. Building owners will suffer lost income and likely be responsible for paying tenant relocation costs, while the tenants will lose sales until they can re-open.

Note that the downtime for the building (and thus the owner) and the tenant may not be the same. Tenants may re-open businesses before construction completes or find alternate space. Accordingly, HAZUS provides the following two definitions:

- 1) Building Recovery Time (for the building)
- 2) Loss of Function Time (for the tenant)

5.5.4.1 Building Recovery Time

The building recovery time is defined as the complete time necessary to restore the building’s intended function. This includes not only the actual construction duration of repair projects but also items such as:

- Detailed building assessments by design professionals
- Design development and preparation of contract documents
- Tendering of construction contracts
- Negotiating with insurance/emergency management agencies for funding
- Review by outside decision-makers (eg. building officials, emergency managers, city heritage advisors)

Under the HAZUS methodology, the building recovery time is only a function of the structural damage state and there are different recovery times for the various occupancies. We have restricted our focus to only retail trade (COM1) occupancy type buildings; the recovery times as listed in HAZUS are shown below:

Table 5.4 – HAZUS Recovery Times

Damage State	Time [days]
None	0
Slight	10
Moderate	90
Extensive	270
Complete	360

Recent experience in Christchurch has shown that as the damage to a community of buildings increases, there may be a “scale effect” on the downtimes. In response to the high concentration of badly damaged buildings throughout the Central business district (CBD), a cordon was set up to prevent public access into the CBD and allow time for building evaluations and decision-making on the recovery efforts. Even one year later, a substantial portion of the CBD remained within the cordon and it was not fully removed until June of 2013 (The New Zealand Herald 2013). Figure 5.16 below shows the extents of the cordon initially (February 2011) and one year later (February 2012).

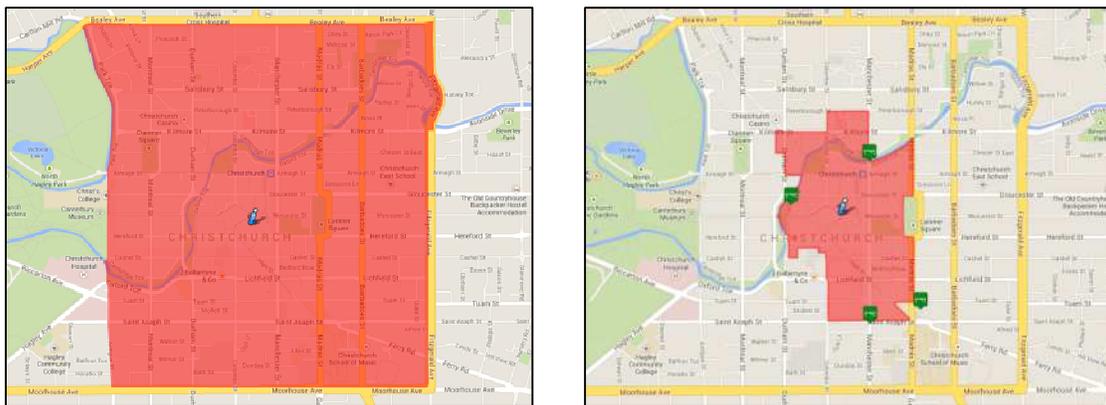


Figure 5.16 – Initial and One-Year Cordon Extents in Christchurch CBD

Source: <http://cera.govt.nz/> (Retrieved July 2014)

Another example is given by Comerio (2006) in which it is shown that the fraction of buildings repaired within 1-2 years rapidly drops off with increasing damage to the overall building stock (as represented by the fraction, or percent, of the building stock closed). Figure 5.17 summarizes the results. Note that the University of California, Berkeley figures are only a scenario estimate, but the trend is clear.

Based on the foregoing discussion, it is concluded that the overall damage to the building stock within a region also has an important impact on building recovery time. To address this issue it was decided to revise the building recovery times to also be a function of the fraction of buildings in an extensive or complete damage state. Figure 5.18 shows the resulting relationship. It can be seen that at up until 10% of the buildings reach a damage state of Extensive or Complete, the downtime is unchanged from that specified in HAZUS; this value was chosen because (as per the fragility curves) it corresponded approximately to the level of shaking experienced by buildings in the Darfield earthquake as well as the various California earthquakes, which resulted in much less long term disruption.

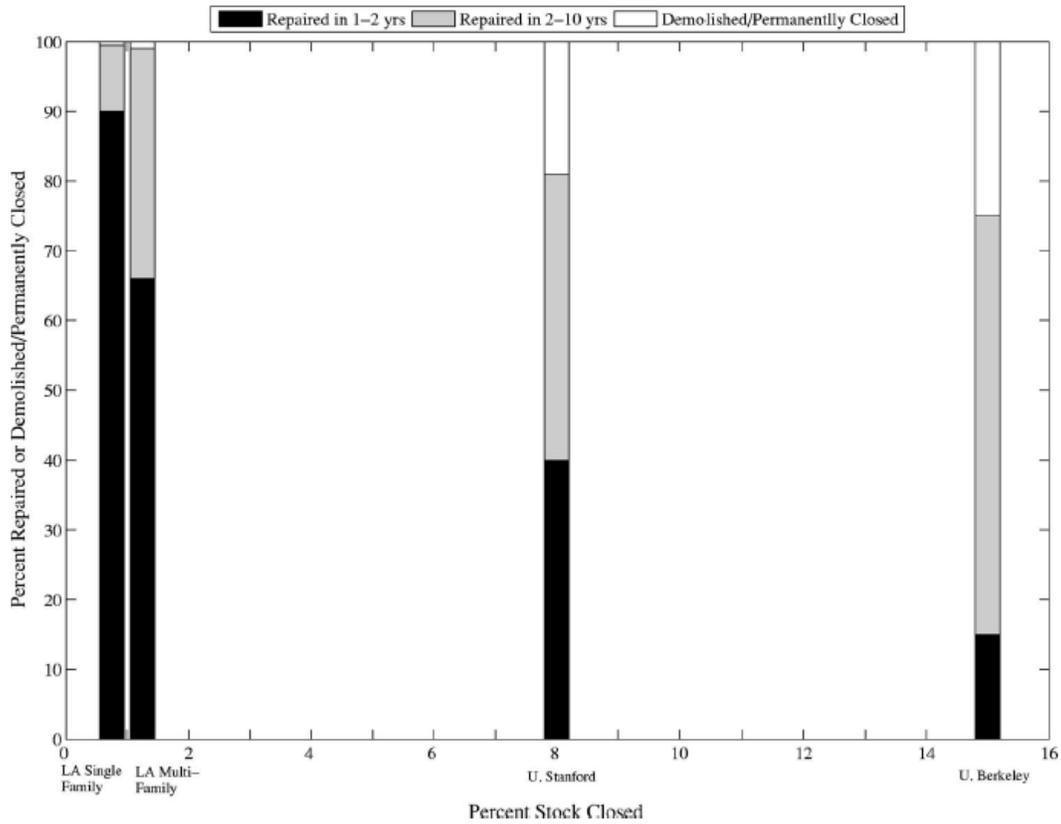


Figure 5.17 – Building Repair Time Distribution vs. Damage
(From Comerio, 2006)

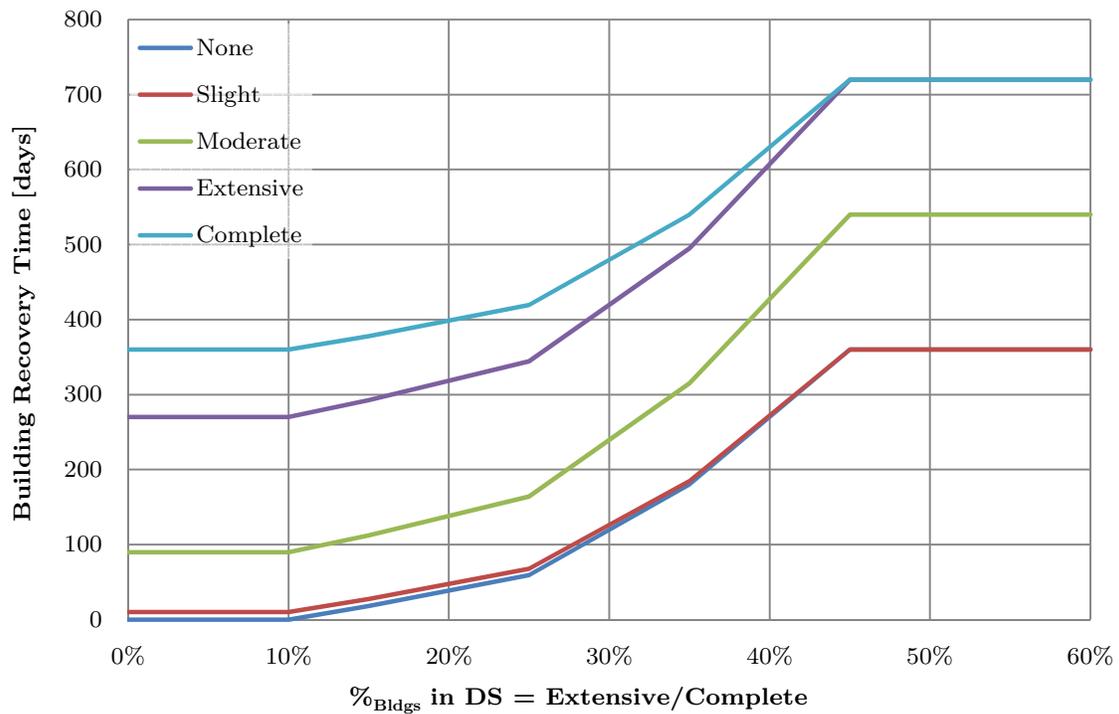


Figure 5.18 – Building Recovery Time Estimates Used in This Study

When 45% of the buildings reach a damage state of Extensive or Complete, the recovery time is set at 360 days for a building in the None or Slight Damage States, 540 days for Moderate damage and 720 for Extensive and Complete; these numbers are intended to approximately correspond to a one year delay, followed by the recovery time. In reality, the recovery time would be affected by other factors such as damage to lifelines and political/insurance issues. Future research is needed to validate and calibrate this model, as a rigorous effort was obviously not completed herein. In passing, however, it should be noted that the added downtime ultimately has a small effect on the cost-benefit analysis, since the requisite ground motions have a low probability of occurrence, even in Victoria, BC.

5.5.4.2 Loss of Function Time

While a given building may take well over a year to be re-occupied (and resume generating rental income), the downtime for businesses are generally much shorter because alternate/temporary locations will be secured or they may re-open before construction is complete. HAZUS recognizes this by specifying a “Loss of Function Time,” which is taken as a fraction of the building recovery time. The HAZUS Loss of Function times were directly used in this study and are as shown in Table 5.5.

Table 5.5 – HAZUS Loss of Function Times

Damage State	Time [days]
None	0
Slight	1
Moderate	9
Extensive	81
Complete	144

5.5.5 Occupant & Pedestrian Exposure

The first key component of the casualty risk is the exposure. Obviously, a URM building that is used strictly for storage and set back well away from any public corridors would pose much less risk of human casualties. In this section we define the occupant and pedestrian densities which represent the indoor and outdoor exposures, respectively.

5.5.5.1 Occupant Density

Two sources were considered for the occupant density: those published in the Canadian Manual for Screening of Buildings for Seismic Investigation (NRC 1993) and the aforementioned San Francisco study by Rutherford and Chekene (1990). HAZUS also

prescribes values, but they rely on census tract information which was not available for this study. The two aforementioned sources specified average occupant densities of 0.0077persons/sq.ft. and 0.0036persons/sq.ft., respectively (accounting for low occupant and empty periods). Ultimately, it was decided to use the values from Rutherford and Chekene because they provided a temporal variation and the values were explicitly intended for loss estimation purposes.

5.5.5.2 Pedestrian Density

Data on pedestrian density was not available in the aforementioned Canadian source and, again, the HAZUS model required census tract information, so it was decided to make use of the values from Rutherford and Chekene (1990), which were specified in terms of lineal feet of building frontage for various areas throughout San Francisco. Victoria is currently significantly less populous than San Francisco was in 1990, although the vast majority of URM buildings are located in the old downtown core ("Old Town", as discussed in Chapter 7). Ultimately, it was felt that the second densest area from the San Francisco study would be the most appropriate representation for a "typical" building in Victoria. The densities in question are repeated in Table 5.6.

Table 5.6 – Pedestrian Densities [persons/1000ft]

Time of Day	10pm-7am	7am-9am	9am-12pm	12pm-1:30pm	1:30pm-5pm	5pm-6pm	6pm-10pm
Density	1	60	20	70	40	120	40

Rutherford and Chekene used these values to make scenario estimates of casualties, while for the cost-benefit analysis completed in this study only an average value is of interest (as the benefits are already averaged in terms of annual expected values). Taking the time weighted average yields a value of 30persons/1000ft, which was used in our study. The foregoing decisions were obviously highly uncertain and refinement of this data through surveys would be valuable.

5.5.6 Casualty Rates

The second major component in estimating casualties is the casualty rates. In the HAZUS methodology, casualty rates are a function of the structural damage states, representing conditional probabilities. Here the two sources considered were HAZUS and Rutherford and Chekene (1990) and again we have indoor (occupant) and outdoor (pedestrian) casualty rates.

5.5.6.1 Occupant Casualty Rates

The HAZUS casualty rates are based on the HAZUS structural damage states. However, the life-safety hazards due to collapse are explicitly recognized by separating the Complete damage state into collapse and non-collapse portions (HAZUS specifies that 15% of URM buildings in the Complete damage state will collapse). Casualty rates are provided for each structural type, although there is little variation other than slightly reduced rates for light frame construction and slightly increased rates for URM. Rather the variability is considered accounted for through the structural fragilities themselves.

Casualties are also separated into four categories, but here we will make the simplified distinction of fatalities and hospitalized injuries. The fatality rates from HAZUS are as specified in Table 5.7.

Table 5.7 – HAZUS URM Fatality Rates

Damage State	Fatality Rate
None	0%
Slight	0%
Moderate	0.001%
Extensive	0.002%
Complete (no collapse)	0.02%
Complete (Collapse)	10%

Rutherford and Chekene (1990) specified fatality rates for different types of URM strengthening as shown below in Table 5.8; the strengthening levels were discussed in Section 5.4.1. The values were based on ATC-13 and calibrated to some extent to the Loma Prieta earthquake, although the higher damage states remain unchanged. Note that the rates are provided in terms of the MDF rather than the individual damage states.

Table 5.8 – Rutherford and Chekene URM Fatality Rates

MDF	Unret.	Bolts-Plus	UCBC	SF 104(f)
0%	0%	0%	0%	Equal to UCBC
5%	0.001%	0.0009%	0.0007%	
20%	0.035%	0.033%	0.023%	
45%	0.35%	0.34%	0.28%	
80%	3.5%	3.5%	3.5%	
100%	20%	20%	20%	

The rates are noticeably different from those of HAZUS: the rate for the Extensive damage state, which we would likely compare with MDF=45%, is noticeably lower for the former. In comparing the Complete damage state, note that the average for the collapse and non-collapse (assuming a 15% collapse rate) is about 1.5% which is the same order of magnitude as the MDF=80% values. The MDF=100% value is much higher but this is not particularly of interest as this represents total destruction of all components and would correspond to about $S_a(1)=4-5g$.

Although Rutherford and Chekene (1990) calibrated their model to the Loma Prieta earthquake (which obviously would have appealed to decision-makers in San Francisco), this is just one event. The HAZUS rates presumably account for the ATC-13 values as well as several actual earthquakes and updated opinions of the experts involved. As such, it was decided to use the HAZUS occupant casualty rates in this study.

5.5.6.2 Pedestrian Casualty Rates

A similar exercise of comparison was undertaken for the pedestrian casualty rates. Table 5.9 shows the HAZUS fatality rates and Table 5.10 shows Rutherford and Chekene’s fatality rates.

Table 5.9 – HAZUS URM Fatality Rates

Structural Damage State	Fatality Rate
None	0%
Slight	0%
Moderate	0.0003%
Extensive	0.0006%
Complete	0.6%

Table 5.10 – Rutherford & Chekene (1990) Pedestrian Fatality Rates

MDF	Unret.	Bolts-Plus	UCBC	SF 104(f)
0%	0%	0%	0%	Equal to UCBC
5%	0.02%	0.0018%	0.0014%	
20%	0.3%	0.28%	0.2%	
45%	7%	6.5%	5.5%	
80%	12%	12%	12%	
100%	15%	15%	15%	

The rates used by Rutherford and Chekene (1990) are again much higher, while the HAZUS rates seem implausibly low. In looking to the events of Christchurch, it is noted that there were 39 fatalities due to URM buildings (as discussed at the outset of Chapter

4). Of these, 35 were pedestrians – i.e. those outside the building (Canterbury Earthquakes Royal Commission 2012). Based on the database provided by the authors, the average street front of the 370 CBD buildings was about 60 feet, for a total of 22,200 feet of frontage. Assuming a pedestrian density of 50 persons per 1000 lin. feet (i.e. 3 persons per 60 feet of frontage) gives about 1100 pedestrians exposed at the time of the earthquake. This translates into a fatality rate of 3%, which compares better with Rutherford and Chekene’s rates. As such it was decided to use these rates in our study.

To be consistent with the remainder of our procedure, it was decided to map the rates from the overall MDF to the HAZUS damage states; this was accomplished by comparing the fatality rates of our model to those listed above at equal overall (not structural) MDF’s. Table 5.11 shows the fatality rates that were used in this study.

Table 5.11 – Pedestrian Fatality Rates Used in This Study

Structural Damage State	Fatality Rate
None	0%
Slight	0.02%
Moderate	0.30%
Extensive	12%
Complete	15%

Unlike the Rutherford and Chekene (1990) study, we have used the same fatality rate regardless of the retrofit status. This is because these rates are associated with the *structural* damage rather than the overall damage. In any case, the adjustments by Rutherford and Chekene were minor and would likely not have any significant impact on our study. The vast majority of the improvement comes from the reduced structural damage at a given seismic demand. Tables 5.12 to 5.15 show the resulting structural damage distributions and overall fatality rates at a variety of seismic demands for each retrofit category.

Table 5.12 – Damage and Pedestrian Fatality Rates for Unretrofitted Buildings

$S_a(1)$	Structural Damage State Distribution					Overall Fatality Rate
	None	Slight	Moderate	Extensive	Complete	
0.12g	53%	38%	9%	0.3%	≅0%	0.1%
0.33g	12%	42%	38%	7%	1%	1.1%
0.70g	2%	17%	48%	24%	9%	4.4%
1.9g	≅0%	2%	21%	35%	42%	11%

Table 5.13 – Damage and Pedestrian Fatality Rates for Braced Parapet Buildings

S _a (1)	Structural Damage State Distribution					Overall Fatality Rate
	None	Slight	Moderate	Extensive	Complete	
0.12g	75%	22%	2.5%	0.3%	≅0%	≅0%
0.33g	25%	48%	25%	1.6%	≅0%	0.3%
0.70g	3%	23%	55%	16%	3%	2.6%
1.9g	≅0%	2%	25%	37%	36%	10%

Table 5.14 – Damage and Pedestrian Fatality Rates for Partial Retrofit Buildings

S _a (1)	Structural Damage State Distribution					Overall Fatality Rate
	None	Slight	Moderate	Extensive	Complete	
0.12g	76%	22%	2%	≅0%	≅0%	≅0%
0.33g	23%	51%	24%	2%	≅0%	0.3%
0.70g	3%	26%	55%	14%	2%	2.2%
1.9g	≅0%	2%	29%	40%	29%	9.2%

Table 5.15 – Damage and Pedestrian Fatality Rates for Full Retrofit Buildings

S _a (1)	Structural Damage State Distribution					Overall Fatality Rate
	None	Slight	Moderate	Extensive	Complete	
0.12g	78%	20%	2%	≅0%	≅0%	≅0%
0.33g	32%	48%	19%	1%	≅0%	0.2%
0.70g	8%	38%	43%	9%	2%	1.5%
1.9g	≅0%	9%	41%	30%	20%	6.7%

In examining the tables, it can be seen that parapet bracing/partial retrofits achieve most of the life safety improvements at reasonably expected levels of shaking (i.e. 0.12g-0.7g). In the 0.7g range, the fatality rates (1.5%-4.4%) are fairly consistent with the previously calculated value for the Christchurch earthquake of 3%. Also, the relative improvement due to parapet bracing from 4.4% to 2.6% (at 0.7g) is fairly consistent with observations from the February 2011 Christchurch earthquake that unrestrained parapets were twice as likely to fail as restrained parapets (Ingham and Griffith 2011b).

5.5.6.3 Accounting for Injuries

Thus far we have accounted only for fatalities in our casualty estimation. Injuries, however, are typically more numerous and can represent a significant portion of the overall casualty losses. HAZUS defines four severities of injuries as follows:

- Severity 1: Injuries requiring medical aid that could be administered by para-professionals (eg. sprains, cuts requiring stitches)
- Severity 2: Injuries requiring a greater degree of medical care and technology (eg. x-rays or surgery), but not expected to progress to life threatening status

- Severity 3: Injuries that pose an immediate life threat if not treated adequately (eg. uncontrolled bleeding, punctured organ)
- Severity 4: Instantaneously killed or mortally injured

Figure 5.19 shows the indoor injury/fatality rates for URM as specified in HAZUS (FEMA 2012) for the “Complete” Damage state. HAZUS specifies separate rates for buildings that do and do not collapse. The rates shown below represent the weighted average of the “Complete (No Collapse)” and “Complete (Collapse)” values, using the HAZUS specified collapse rate of 15% for URM buildings (i.e. 15% of buildings in the Complete damage state are assumed to collapse). Also note that “Severity 4” represents fatalities, as shown in Table 5.7 (i.e. $.15*10\%+.85*.02\%=1.52\%$).

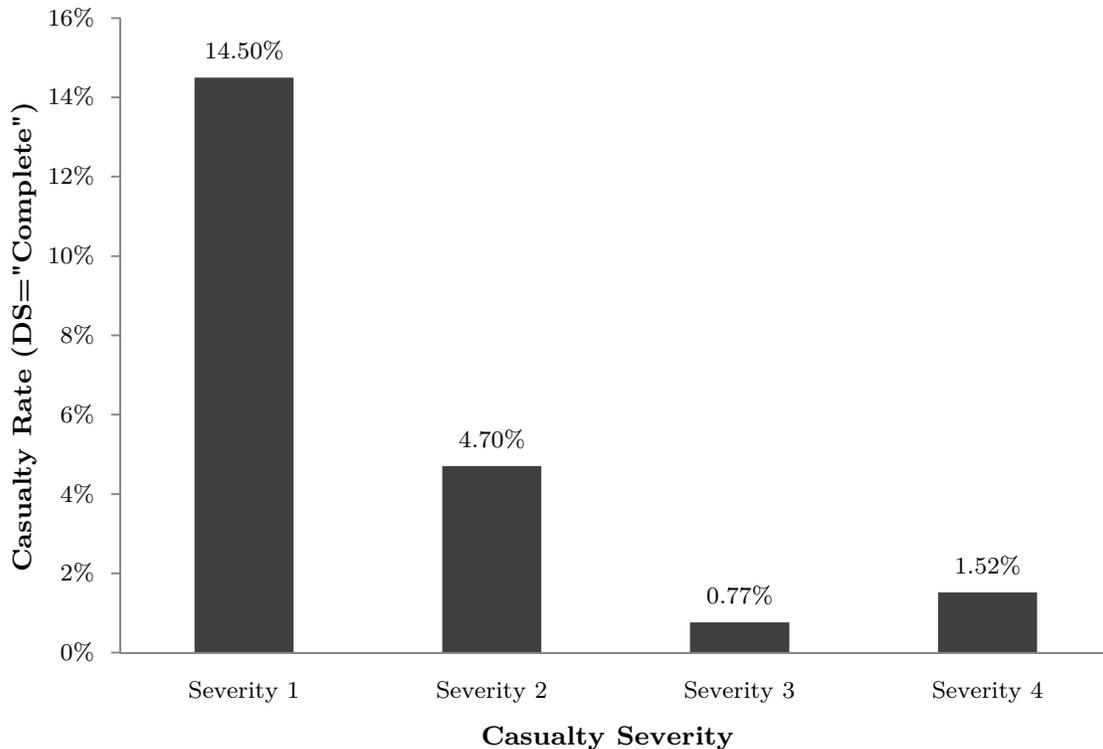


Figure 5.19 – HAZUS Casualty Rates for URM for “Complete” Damage

Because the outdoor fatality rates used in our study are not from HAZUS, we do not have this level of refinement available. Instead, we will take the same approach as Rutherford and Chekene (1990) and other past works and estimate the number of hospitalized injuries as four times the number of fatalities (Steinbrugge et al 1972, ATC 1985). We will apply this to both the indoor and outdoor fatalities. However, we will make use of the above noted distribution in monetizing the injuries.

5.5.6.4 Monetizing Casualties

In order to include the casualties in an overall cost-benefit ratio, we must translate them into monetary terms. Although highly contentious in terms of the values used and methodologies of determining these values, it is typical practice in policy-making on safety improvements such as environmental issues and transportation (Posner and Sunstein 2004, Miller 2000).

A relatively comprehensive review of the literature and recommendations was recently published by the US Department of Transportation (USDOT 2013). This document was developed specifically to provide guidance to analysts on the Value of a Statistical Life (VSL) for use in cost-benefit analyses. Since results in the literature were found to change significantly with time (the VSL increases with income) and the results for Canada and the US were found to be similar (Miller 2000), the US Department of Transportation values were deemed to be suitable for use in our study. Note that FEMA has previously provided guidance on defining the VSL (FEMA 1992, 1994), but that these documents have become outdated. The USDOT specifies values as follows in 2012 US dollars:

- Best Estimate: \$9.1 million
- Lower Bound: \$5.2 million
- Upper Bound \$12.9 million

It is recommended that these values be considered separately so as to consider the range of uncertainty rather than taking a weighted average and so this will be the approach taken herein. Minor adjustments could have been made in converting to 2014 Canadian dollars, but such adjustments were deemed unnecessary given the level of uncertainty inherent in VSL.

The document also provides VSL percentages to be used to monetize injuries. The injuries are classified in terms of severity as per the Abbreviated Injury Scale (TARN 2014), as shown below:

- AIS 1 (Minor) = 0.3%*VSL
- AIS 2 (Moderate) = 4.7%*VSL
- AIS 3 (Serious) = 10.5%*VSL
- AIS 4 (Severe) = 26.6%*VSL
- AIS 5 (Critical) = 59.3%*VSL
- AIS 6 (Unsurvivable) = 100%*VSL

For this study, it is the value of a “hospitalized injury” that is of interest. In terms of the HAZUS severities, this would include Severities 2 and 3. Based on a comparison of the descriptions of the HAZUS severities and the AIS (Abbreviated Injury Scale) classifications, the following equivalencies and average VSL values were assigned:

- HAZUS Severity 1 = AIS 1 (0.3%VSL)
- HAZUS Severity 2 = AIS 2/3 (7.6%VSL)
- HAZUS Severity 3 = AIS 4/5 (43%VSL)
- HAZUS Severity 4 = AIS 6 (100%VSL)

Severity 4 casualties (i.e. fatalities) have already been accounted for separately and, thus, are of no further interest. Based on the distribution of injuries in Figure 5.19, it can be seen that Severity 2 injuries are approximately six times more common than Severity 3 injuries. Taking the weighted average of these two yields an average of 13%VSL, which was rounded up to 15%VSL to account for non-hospitalized injuries. This latter figure was used throughout the cost-benefit analysis.

5.5.7 Scenario versus “Expected” Losses

An important distinction for loss estimates is that between scenario and “expected” losses. The former gives a deterministic estimate of losses conditioned on a given seismic demand (i.e. a scenario earthquake) while the latter is a probabilistic value. Typically, expected losses are calculated in terms of annual expected values, as this is a convenient measure of seismic hazard, and then converted a net present value for comparison to retrofit costs. Scenario loss estimates have the benefit of being more tangible for decision-makers and the public. Conversely, expected losses are not as tangible but are the more meaningful measure of risk and are a better basis for decision-making. In this study we deal primarily in terms of expected losses.

To calculate the expected losses, we multiply the damage for a given level of shaking by the probability of occurrence for that shaking. This is accomplished numerically by discretizing the hazard curve into small segments and performing the multiplication over the entire range of significant hazard, illustrated by the following example. Note that the figure shows only the MDF (i.e. building losses) curve, but the procedure is similar for the casualties and downtime related losses.

$$EAL = \Sigma[(Loss\ Value\ | \ Level\ of\ Shaking = i) * (Annual\ Prob.\ of\ shaking = i)] \quad (5-2)$$

Where:

EAL ≡ Expected Annual Loss

Note that for Victoria’s seismic hazard (one of the highest in Canada) we found that hazards beyond about $S_a(1)=2g$ were small enough to have a negligible impact on the risk and so this was the cut-off used for our analysis.

The following example shows the steps involved in determining the expected losses:

- 1) Identify the seismic hazard. In this study, we use the $S_a(1)$ hazard curve for Victoria. The hazard curve is then discretized into segments throughout the range of shaking that materially impacts the results (in this case, 0-2g). Figure 5.20 shows the curve with a single discretized segment. For each segment, the result is an average shaking intensity ($S_a(1)_{AVG}$) and a corresponding annual probability of occurrence ($\Delta\lambda$). In Figure 5.20, we see that $\Delta\lambda=.00003$ for $IM_{AVG}=1.0g$
- 2) Identify a building with a given overall value and motion-damage relationship. A replacement value of \$260/sq.ft. is used in this study (as discussed in Section 5.5.2). For the purposes of this example, we will assume a floor area of 8000sq.ft., yielding an overall replacement value of \$2.08million. Recall that the overall motion-damage relationship was derived in Section 5.5.3 based on relationships for each of the four individual components (structural, drift-sensitive NSC, acceleration-sensitive NSC, and contents) and an assumed breakdown of these components (see section 5.5.2). The overall motion-damage curve for an unretrofitted building is shown in Figure 5.20 (repeated from Figure 5.15).
- 3) For each IM_{AVG} of the hazard curve, there is a corresponding level of damage (i.e. loss) on the motion-damage curve. In Figure 5.20, we can see that a damage ratio of 60% (i.e. a loss equal to 60% of the replacement value) corresponds to $S_a(1)_{AVG}=1.0g$.⁹
- 4) The expected annual loss for the given segment of the hazard curve can then be calculated as:

$$EAL_{S_a(1)=1.0g} = 60\% * \$2.08\text{million} * .00003 = \$37.44/\text{year}$$

- 5) This procedure is then repeated for each segment of the hazard curve and summed as per Equation 5-2 to yield the total EAL

⁹ A damage ratio of 60% exceeds the previously defined ECLR of 50% and so the, and so the resulting loss value assigned in this instance would be 100% (i.e. the building would be replaced). However, the purpose of the forgoing example was to demonstrate the overall process.

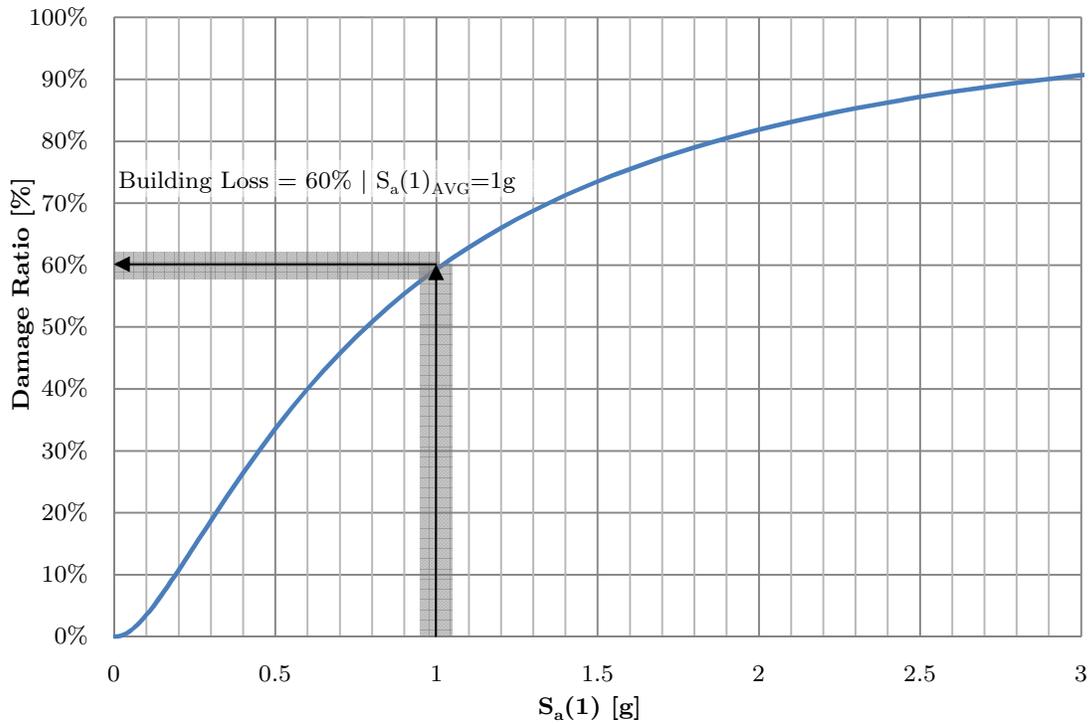
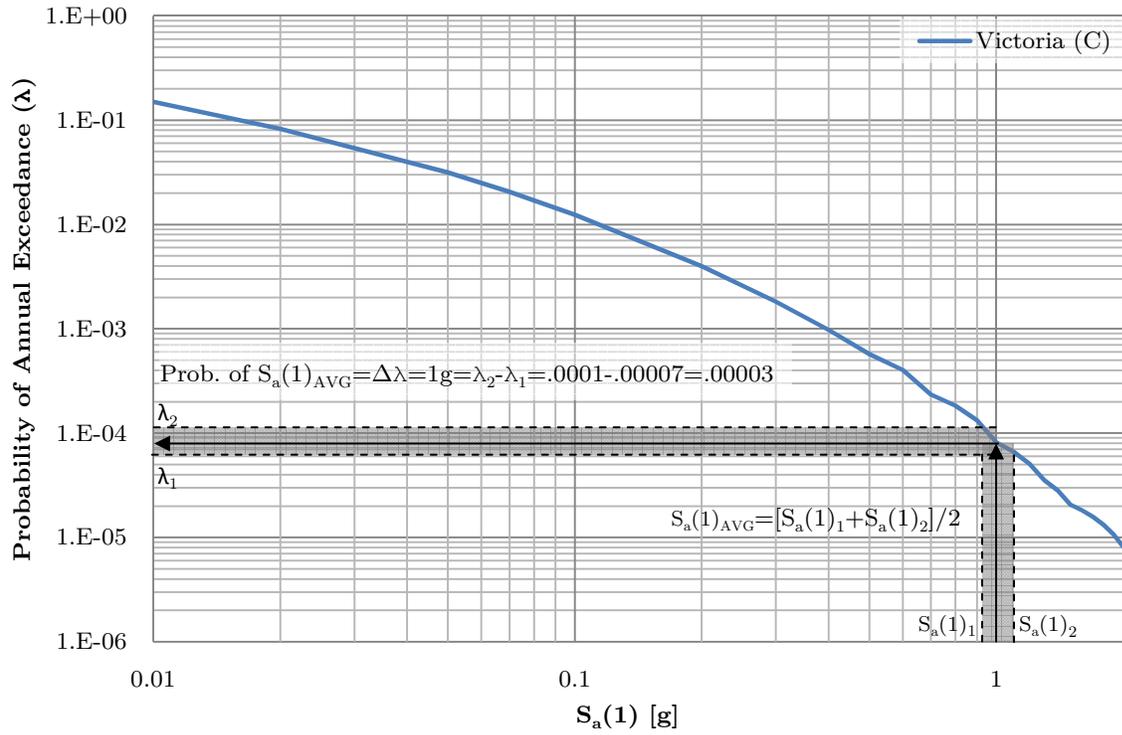


Figure 5.20 – Hazard Curve Discretization (Top) and Damage Curve (Bottom)

5.5.8 Economic Parameters

With the annual expected losses calculated, we must convert these values to a present value for eventual use in our cost-benefit analysis. There are two key parameters involved in this process as enumerated below and subsequently discussed.

- 1) Time Horizon
- 2) Discount Rate

5.5.8.1 Time Horizon

The time horizon is the number of years over which the losses (and thus benefits in the form of reduced losses) are to be accumulated. FEMA (1994) notes that time horizons of 30 to 50 years are typical for building projects. In the Seattle study (Gibson Economics 2014), the lower value of 30 years was chosen as the “base” case and an even shorter period was used for a lower bound – the rationale for this decision was that owners may sell or replace their URM buildings before 30 years. However, this neglects the fact that the market value of the building would presumably be higher than if it were left unretrofitted; moreover, many URM buildings are heritage buildings and it is quite unlikely they would be permitted to be replaced. Finally, a portion of the benefit lies in the improved life safety and the sale of the building would have no effect on this loss.

For the aforementioned reason, a base case time horizon of 50 years will be adopted in this study.

5.5.8.2 Discount Rate

Discount rates are applied to annual expected losses (in future years) and the result is to place more value on losses (or benefits) that occur sooner. The rationale for discounting, as stated in Circular A-4 from the United States Government Office of Management and Budget (OMB 2003) is as follows:

- 1) *Resources that are invested will normally earn a positive return, so current consumption is more expensive than future consumption*
- 2) *Postponed benefits also have a cost because people generally prefer present to future consumption.*
- 3) *Also, if consumption continues to increase over time, as it has for most of U.S. history, an increment of consumption will be less valuable in the future than it would be today, because the principle of diminishing marginal utility implies that as total consumption increases, the value of a marginal unit of consumption tends to decline.*

Principle #1 is easily applied to our situation in that rather than spending money on seismic retrofitting today, one could have invested this money elsewhere with a certain return (which would effectively reduce future losses). As noted in Circular A-4, the average rate of return on capital in the United States (which we presume here is applicable to Canada) is 7% and this is the recommended discount rate for private investment decisions. FEMA 227 (FEMA 1992) recommends discount rates of 3-6%. FEMA 255 (FEMA 1994), which focuses on federal buildings, recommends a 4% discount rate, based on the rate of return for treasury notes.

Principles #2 and #3 are less clear cut: they are valid for any given individual, but ethical issues arise when dealing with losses/benefits accrued to future individuals and this may play into an individual's time preference. Circular A-4 recognizes this and states that for intergenerational discounting, a reduced discount rate of 1-3% is appropriate.

In the Seattle study, a base discount rate of 7% was used for all losses/benefits, with 3% and 10% rates used for sensitivity analyses. However, FEMA 227 states that *“using discount rates as high as the 10% discount rate [formerly] mandated by the Office of Management and Budget...will produce unreasonable results.”*

In this study we will use the following discount rates:

- For owner losses/benefits, a best estimate of 5% and lower and upper bounds of 3% and 7%, respectively
- For life-safety losses/benefits, a best estimate of 3%, with lower and upper bounds of 1% and 5%, respectively

5.6 Quantifying The Losses

This section presents the results of the loss estimates carried out in accordance with the previous section. Refer to Section 5.5 for detailed discussion on the methodology. Losses are presented in terms of their present value of the annual expected losses (henceforth referred to as “expected losses”), using the best estimate economic parameters of Section 5.5.8. Note that the values are based on a building in Victoria of 8000sq.ft. in area, with approximately 30 feet of pedestrian exposure (i.e. sidewalks and alleys). A change in area would have no effect on the subsequent cost-benefit results, but a change pedestrian exposure or seismic hazard would. Results are provided for buildings located on Site Class B, C, D, and E soils. The values presented in this section are for the base case parameters. A sensitivity analysis for many parameters is presented in Section 5.10.

5.6.1 Building Damage Losses

Building damage losses were calculated for each of the four strengthening statuses. They include the use of an economic critical loss ratio, as discussed in Section 5.5.3.5. Table 5.16 shows the results for the base case. The precision of these values should not be taken to imply commensurate accuracy, but it was decided not to round the output results, primarily for ease of back-checking during thesis preparation.

Table 5.16 – Building Damage Losses

Site Class	P.V. of Expected Losses [\$] by Strengthening Status			
	Unretrofitted	Braced Parapet	Partial Retrofit	Full Retrofit
B	\$31,689	\$25,339	\$20,474	\$14,147
C	\$59,935	\$49,174	\$39,029	\$26,992
D	\$89,479	\$74,783	\$59,887	\$40,885
E	\$161,125	\$136,233	\$113,167	\$78,930

5.6.2 Downtime-Related Losses

In this study the downtime related losses are considered from a building owner’s perspective and include tenant relocation expenses and lost rental income. Tables 5.17 and 5.18 show the results for the base case for tenant relocation expenses and lost rental income, respectively.

Table 5.17 – Tenant Relocation Expenses

Site Class	P.V. of Expected Losses [\$] Strengthening Status			
	Unretrofitted	Braced Parapet	Partial Retrofit	Full Retrofit
B	\$514	\$315	\$281	\$215
C	\$981	\$556	\$490	\$384
D	\$1,569	\$895	\$788	\$610
E	\$2,994	\$1,842	\$1,637	\$1,258

Table 5.18 – Lost Rental Income

Site Class	P.V. of Expected Losses [\$] Strengthening Status			
	Unretrofitted	Braced Parapet	Partial Retrofit	Full Retrofit
B	\$2,502	\$1,327	\$1,226	\$1,065
C	\$4,855	\$2,527	\$2,344	\$2,027
D	\$7,356	\$3,871	\$3,578	\$3,138
E	\$13,912	\$8,319	\$7,728	\$7,176

5.6.3 Casualty-Related Losses

Casualties were calculated separately for indoor and outdoor occupants. They include both fatalities and injuries and were monetized using the VSL's provided in Section 5.5.6.4. Tables 5.19 and 5.20 show the expected losses for indoor and outdoor casualties, respectively. Note that although the casualty rates were much higher for outdoor occupants, the actual casualty-related losses are similar; this is because of the relatively low number of occupants outside a given building as compared to inside (on a probabilistic basis).

Table 5.19 – Indoor Casualty Losses

Site Class	P.V. of Expected Losses [\$] Strengthening Status			
	Unretrofitted	Braced Parapet	Partial Retrofit	Full Retrofit
B	\$9,816	\$3,954	\$2,530	\$2,291
C	\$17,681	\$7,694	\$5,087	\$4,414
D	\$25,574	\$10,755	\$7,112	\$6,270
E	\$67,481	\$32,767	\$22,826	\$18,887

Table 5.20 – Outdoor Casualty Losses

Site Class	P.V. of Expected Losses [\$] Strengthening Status			
	Unretrofitted	Braced Parapet	Partial Retrofit	Full Retrofit
B	\$10,231	\$4,853	\$3,996	\$3,033
C	\$17,927	\$8,512	\$7,057	\$5,429
D	\$27,644	\$12,618	\$10,393	\$8,105
E	\$61,085	\$30,830	\$25,871	\$19,628

5.6.4 Total Expected Losses

The total expected losses to the building owner (damage, relocation costs, lost rental income) and to the general public (indoor and outdoor casualties) are shown graphically in Figure 5.21. Only results for Site Classes C and E are shown, for illustrative purposes. Recall that these are the probabilistically-derived values and do not represent the losses from any specific event. It is interesting to note that, for Site Class E, the public losses (casualties) represent a greater fraction of the total losses. Also, it can be seen that the Site Class E losses are not simply equal to the Site Class C losses multiplied by a code-specified foundation factor. This is due to nonlinearities in the hazard curve as well as the motion-damage and damage-casualty/downtime relationships.

The difference between any strengthening case and the unretrofitted case represents the reduced expected losses. This is the benefit of the seismic strengthening measure, as will be used in cost-benefit analysis of Section 5.9.



Figure 5.21 – Total Expected Losses for Victoria Site Class C (Top) and E (Bottom)

5.6.5 Other Losses Not Considered

A variety of other potential losses were not quantified in our analysis. In some cases this was because they were highly intangible, such as lost historic fabric or indirect economic losses (eg. due to decreased tourism). Both of these could be significant.

The city of Victoria is known for its heritage and places a certain value on it, as evidenced by the Building and Tax Incentive Programs, as discussed in Chapter 3. In the wake of the Christchurch earthquake, over 242 of 252 heritage buildings inspected by the Canterbury Earthquake Recovery Authority were either partially or wholly demolished (and of these, 199 were wholly demolished) (CERA 2014). Tourism is another important part of Victoria's economy. Note that in the year following the February 2011 earthquake, Christchurch suffered a loss of tourism revenue of \$235million. This represents roughly 10% of the entire province's tourism revenue of about \$2.3billion (University of Auckland 2012), of which Christchurch represents some significant fraction. Of course, the widespread damage to URM buildings was only one of many contributing factors to this loss.

5.7 Expected Benefits of Strengthening

With the losses quantified for each case, the benefits for each level of strengthening are readily obtained as the difference in expected losses. The benefits are derived by examining the expected losses and are again separated into building owner and public benefits. Table 5.21 shows benefits by strengthening measure by site class.

Table 5.21 – Present Value of Expected Benefits for Seismic Strengthening

Site Class	Type of Benefit	Strengthening Status		
		Braced Parapet	Partial Retrofit	Full Retrofit
B	Building Owner	\$7,724	\$12,724	\$19,278
	Public	\$11,240	\$13,521	\$14,722
	Total	\$18,963	\$26,245	\$34,000
C	Building Owner	\$13,516	\$23,909	\$36,370
	Public	\$19,403	\$23,463	\$25,765
	Total	\$32,918	\$47,372	\$62,135
D	Building Owner	\$18,855	\$34,152	\$53,770
	Public	\$29,844	\$35,713	\$38,843
	Total	\$48,699	\$69,865	\$92,614
E	Building Owner	\$31,636	\$55,499	\$90,667
	Public	\$64,969	\$79,869	\$90,051
	Total	\$96,605	\$135,369	\$180,718

Some interesting trends arise when examining the results. Firstly, it can be seen that although "building owner" (building damage, tenant relocation costs, and lost rental costs) losses were consistently higher than the "public" losses (casualties) for a given strengthening level, this is not always true for the benefits. This is because the majority of the life-safety benefits are achieved at the braced-parapet level of strengthening: Figure 5.22 shows the benefits achieved for each strengthening level, relative to a Full Retrofit. As can be seen, parapet bracing offers significant benefits in terms of life-safety, and only modest benefits in terms of reduction of damage and downtime.

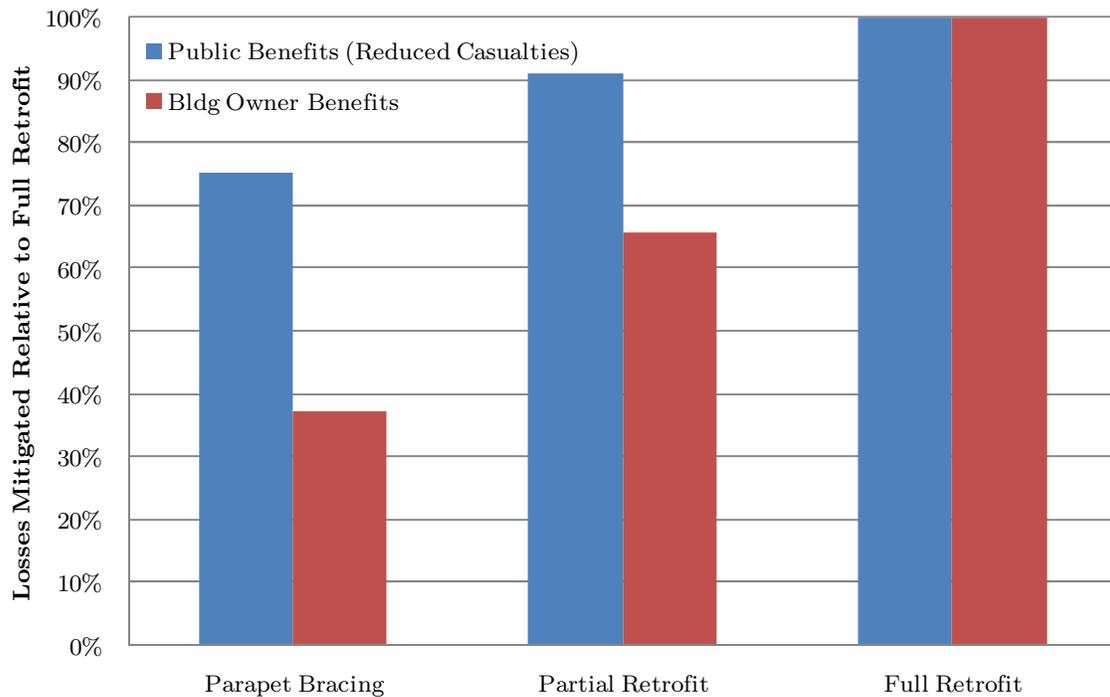


Figure 5.22 – Relative Benefits by Strengthening Level

Another item worthy of discussion is the breakdown of the benefits by loss type: this could perhaps provide the basis for cost sharing between building owners and the public. Figure 5.23 shows the relative contributions of loss reduction for the three levels of strengthening for Site Class C. The vast majority of the benefits lie in reduced casualties and building damage. As aforementioned, life-safety benefits become an even larger fraction for softer soils (site class D and E).

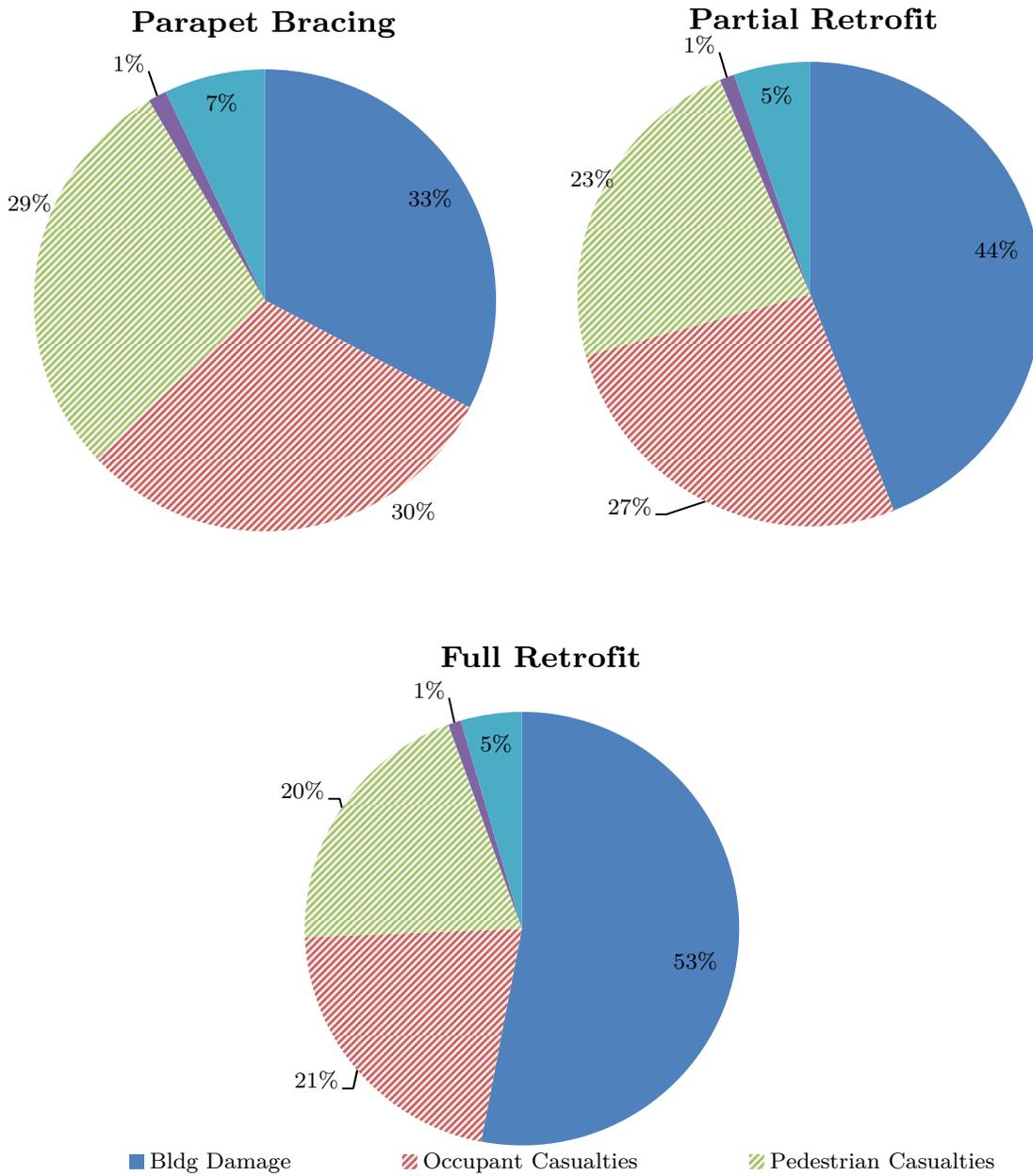


Figure 5.23 – Breakdown of Loss Reductions by Strengthening Level (Site Class C)

5.8 Quantifying the Costs of Seismic Upgrading

In order to perform the cost-benefit analyses, we must also quantify the costs associated with each strengthening measure. The most significant cost is the design and construction cost. Another possible cost would be the cost of disruption to tenant businesses and lost rental income for the building owner. Such costs would only be incurred if the performing the work required that the building be vacated, which is more typical of full retrofits. Another "cost" often discussed as resulting from seismic retrofitting is increased rental rates; however, as noted by Martin Jenkins Associates (2012), this is simply a portion of the retrofit costs being passed on to tenants and so including such a cost would be "double counting" the retrofit cost.

The following sections briefly present the methodology and results of the cost analyses for each strengthening measure. Appendix A contains more detailed information including a review of the available literature. All costs are provided in 2014 Canadian dollars (including outdated/international costs which have been converted) and are in terms of a unit cost per square foot of gross floor area. While other unit measurements (eg. lineal feet of parapet) are more appropriate in some cases, normalizing all costs by the same unit measurement was convenient and sufficiently accurate for our purposes (i.e. a cost benefit analysis for a "typical" building). These unit costs are not necessarily appropriate for any single building and should not be used as such. Appendix A contains the original unit costs, derived in their appropriate unit measurements.

5.8.1 Parapet Bracing

Parapet bracing is obviously the least costly of the three strengthening measures. A cost analysis by Wong (1987) indicated a cost of \$1.70/sq.ft., although it included just the cost of the anchors and structural steel related work (eg. roofing and sheathing work were included elsewhere).

Detailed cost estimates were also completed by the author, by applying the results of a component-by-component estimate to 12 example buildings in Victoria. The estimates indicate costs as shown in Table 5.22. The median cost will serve as our base case.

Table 5.22 – Costs for Parapet Bracing

Low	Base	High
\$2/sq.ft.	\$3/sq.ft.	\$6/sq.ft.

Note that the variation is simply the result of applying the same parapet bracing unit rates (in lineal feet of parapets) to various buildings. Additional variation due to more or less expensive design details or fluctuating market conditions would be expected for actual projects. Factors that affected the range included the shape of the building and presence of "tall" parapets (which require structural steel bracing) versus "short" parapets (which require only anchor bolts). In our analysis, we essentially accounted for tying all walls and roof together with "short" parapet type of work, even in the case of, say, a roof with cornices but no parapets.

It should also be noted that this cost included targeted repointing, re-roofing, and localized replacement of roof sheathing. An allowance of 15% for soft costs (eg. consulting and permit fees) was also included.

5.8.2 Partial Retrofitting

Partial retrofits would typically include the aforementioned parapet bracing work, but also providing similar anchors at all wall-floor intersections and possibly out-of-plane strengthening elements such as strongbacks.

Rutherford and Chekene (1990) provided costs for a "bolts-plus" type partial strengthening measure; this is a fairly comprehensive partial strengthening scheme which includes tension anchors, shear anchors, and out-of-plane strengthening. The cost indicated for the type of building most similar to our typical (commercial occupancy) building ranged from \$17-28/sq.ft.

The author also completed cost estimates, again based on detailed component estimates applied to buildings in Victoria. Note that the estimates included only the above noted parapet work, plus tension ties at all floors. The costs used in this study were based primarily on these estimates and are shown in Table 5.23. These values are for thru-bolted anchors; epoxy anchors (as are popular in Victoria) would be more costly (see Appendix A).

Table 5.23 – Costs for Partial Retrofits

Low	Base	High
\$6/sq.ft.	\$10/sq.ft.	\$14/sq.ft.

The costs above are considerably lower than the values given by Rutherford and Chekene (see Table 5.3). However, lower costs would be expected given that the work (just parapet bracing and tension ties) represents a reduced scope of work. A more

appropriate comparison may be to FEMA 156 (FEMA 1988), which indicates a cost of \$6-9/sq.ft. for "partial compliance" to the City of Los Angeles' retrofit ordinance, Division 88, which this required just parapet bracing and tension anchors.

The costs for partial retrofitting are highly variable, but this is expected since the scope of work is inherently unclear and variable. The question then becomes: what cost (and scope of work) most appropriately represents our loss estimates? It was felt that the "bolts-plus" type of strengthening was likely beyond what was, on average, present in the buildings upon which our damage statistics from Chapter 4 were based, as the partially retrofitted building samples came from the Los Angeles area as well as New Zealand. Los Angeles specifically did *not* require out-of-plane strengthening to be undertaken to classify as partially strengthened and the New Zealand standards do not explicitly specify whether or not out-of-plane strengthening is required. Accordingly, as discussed in Section 4.9.3, we accounted for only limited benefits in partial retrofitting over braced-parapet buildings. As such, it was deemed that the values from our estimates were more appropriate and these values were used in our study.

5.8.3 Full Retrofitting

Full retrofits typically include the aforementioned partial strengthening measures as well as remaining aspects such as out-of-plane strengthening (eg. strongbacks, if not included in partial strengthening), in-plane strengthening of walls (eg. structural steel braced frames) and diaphragm strengthening. There are many possible combinations and levels of strengthening measures, so component-by-component detailed estimates were not feasible.

Fortunately the industry sponsor (VCHT) was able to provide cost information on actual past seismic upgrading projects in Victoria, which received funding under the City of Victoria's Tax Incentive program (TIP), as discussed in Chapter 3. Such data is likely the best possible source of cost information since inherently includes local design and construction preferences and market conditions for Victoria. Furthermore, cost data was explicitly broken out into "seismic" and "non-seismic" portions, since the TIP tax exemption is based on the cost of the seismic upgrading, rather than the entire project.

With the aforementioned cost data, it was possible to derive the cost of seismic upgrading per square foot of gross floor area for each building and fit a probability distribution to the data. There were 23 sample seismic upgrading projects in total. Figure 5.24 shows the histogram and corresponding lognormal distribution.

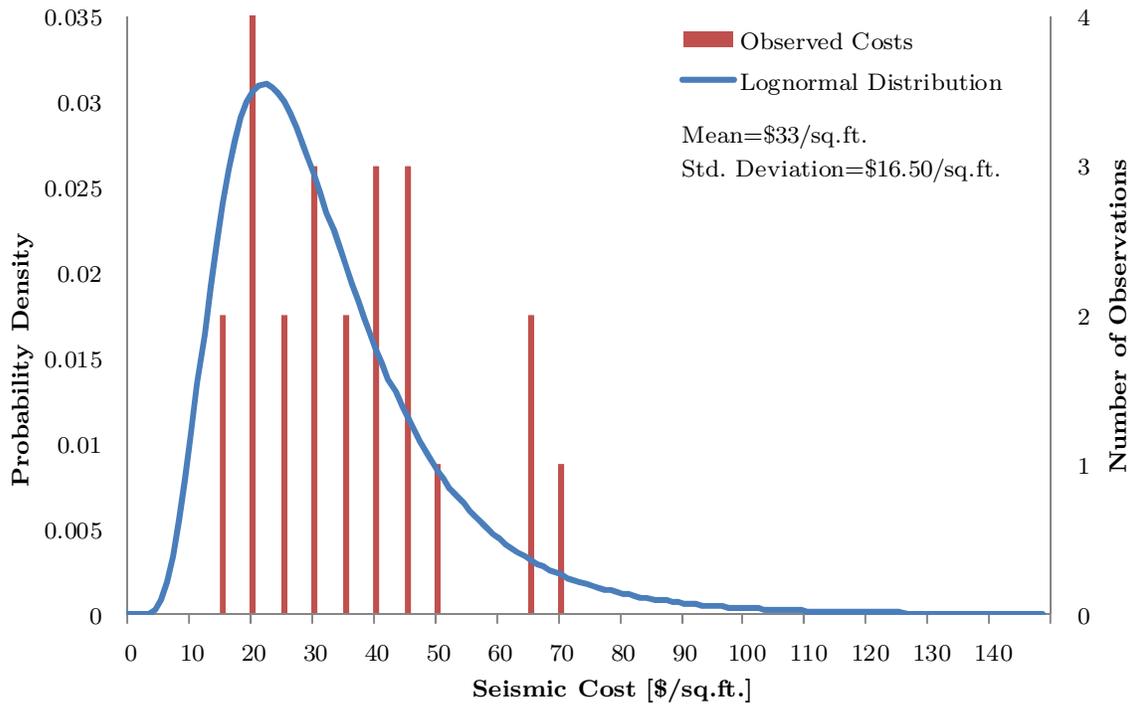


Figure 5.24 – Full Retrofit Costs from Victoria Projects

For our study, we will use the mean cost as our base case and ± 1 standard deviation values as our high and low values for the sensitivity analysis. Thus, the costs are as follows:

Table 5.24 – Costs for Full Retrofits

Low	Base	High
\$17/sq.ft.	\$33/sq.ft.	\$50/sq.ft.

5.8.4 Other Costs Not Considered

Although not included in our cost-benefit analysis, there are a few other potential “costs” worth discussing. Firstly, if mandatory retrofitting were considered, past experience has shown that a number of owners may choose to demolish their buildings rather than strengthen them, which would represent a loss of historic fabric. There are methods of estimating owners’ abilities to comply with mandatory retrofit laws and they are discussed in Recht Hausrath & Associates (1990, 1993). Based on a comparison of retrofit costs to income levels for commercial occupancy buildings in San Francisco (in 1990), Recht Hausrath estimated the following:

- Under a “bolts-plus” type level of mandatory retrofitting, about 92% of buildings would be retrofitted and 8% of buildings would be at risk of being demolished

- Under a UCBC level of retrofitting (similar to practice in Victoria), 90% would eventually comply (although owners would likely delay as long as possible) and 10% would be at risk of demolition
- Under a mandatory retrofit level meeting the San Francisco code requirements (SFBC section 104f), only 80% of buildings would eventually comply and 20% would be at risk of demolition

Although not considered here, owners of residential URM buildings had significantly less ability to comply with retrofits, due to limitations on rent increases and costs due to disruption. It is also worth noting that demolitions of residential URM buildings resulting from retrofit requirements tend to increase rental rates due to a reduced supply of rental stock.

5.9 Cost-Benefit Analysis

With the benefits defined in Section 5.7 and the costs defined in Section 5.8, we can proceed with the cost-benefit analysis. This section presents the results of our base case and section 5.10 provides a sensitivity analysis. The results are derived for Victoria buildings and they may not necessarily be representative of other regions. The most crucial factor is the seismicity. As aforementioned, the cost-benefit analysis is based on an 8000sq.ft. building (most likely 2 or 3 stories), with 30 lineal feet of streetfront exposure in a downtown location with an average (for downtown) amount of pedestrian activity. Results are provided for site classes B through E. See Appendix E for calculations.

5.9.1 Overall Cost-Benefit

The overall benefit/cost¹⁰ ratio (BCR) includes all of the quantified benefits and represents whether or not the project is a good investment (without regard for where the funding comes from). A BCR greater than 1.0 indicates that the project is an economically viable investment, although it should be remembered that a significant fraction of these benefits represent reduced casualties. Table 5.25 provides the benefits (B), costs (C), and the BCR.

¹⁰ Throughout this thesis, the term "cost-benefit" (or "cost/benefit") has typically been employed. For the analysis results, however, the order is reversed (i.e. the results are benefits divided by costs). Except for this one instance, it was decided not to reverse the in-text terminology, as "cost-benefit" appeared to be more commonly used in the literature.

Table 5.25 – Overall Benefit/Costs Results

Site Class	Benefits/Costs [\$P.V.] and BCR								
	Braced Parapet			Partial Retrofit			Full Retrofit		
	B	C	BCR	B	C	BCR	B	C	BCR
B	\$18,963	\$24,000	0.79	\$26,245	\$80,000	0.33	\$34,000	\$264,000	0.13
C	\$32,918	\$24,000	1.37	\$47,372	\$80,000	0.59	\$62,135	\$264,000	0.24
D	\$48,699	\$24,000	2.03	\$69,865	\$80,000	0.87	\$92,614	\$264,000	0.35
E	\$96,605	\$24,000	4.03	\$135,369	\$80,000	1.69	\$180,718	\$264,000	0.68

While it could be argued that retrofits for buildings on softer soils may differ from on firmer soils, it was decided not to pursue this detail, because the cost is much more dependent on scope than on exact force level and the fact that full retrofits could not achieve a $BCR > 1$ as-is meant that an increase in cost would not change the conclusions regarding the economic viability.

Based on these results, parapet bracing appears to be a worthwhile upgrading measure for the majority of buildings; although the BCR for site class B falls slightly below unity, recall that we have not included every possible cost (including heritage preservation). On softer soils (site classes D and E), further upgrading in the form of “partial retrofits” may be justified. Recall that “partial retrofits” were taken to be simply tension ties at all walls/floors and it was assumed that through-bolted anchors could be used throughout (which are less costly than epoxy anchors, as shown in Appendix A). Full retrofits, however, appear to be not economically justified.

5.9.2 Owner-Only Cost-Benefit

The owner-only BCR includes just the benefits due to reduced building damage, tenant relocation expenses, and lost rental income. Table 5.26 provides the owner only benefits, the costs, and the owner-only BCR’s. Note that the costs remained unchanged.

Table 5.26 – Owner-Only Benefit/Costs Results

Site Class	Benefits/Costs [\$P.V.] and BCR								
	Braced Parapet			Partial Retrofit			Full Retrofit		
	B	C	BCR	B	C	BCR	B	C	BCR
B	\$7,724	\$24,000	0.32	\$12,724	\$80,000	0.16	\$19,278	\$264,000	0.07
C	\$13,516	\$24,000	0.56	\$23,909	\$80,000	0.30	\$36,370	\$264,000	0.14
D	\$18,855	\$24,000	0.79	\$34,152	\$80,000	0.43	\$53,770	\$264,000	0.20
E	\$31,636	\$24,000	1.32	\$55,499	\$80,000	0.69	\$90,667	\$264,000	0.34

In examining the preceding table, we can see that even a simple parapet bracing upgrade is likely not a good investment strictly from a building owner’s point of view. However, on very poor soils, a parapet bracing upgrade may be justified. In any case, it provides evidence for cost-sharing among the public and building owners.

The results also suggest that, in any type of mandatory seismic upgrade situation where owners are expected to entirely bear the costs, it would be in their interest to delay as long as possible and perhaps choose not to comply. The Seattle study (Gibson Economics 2014) provides further discussion on decision-making processes for building owners.

5.9.3 Additional Considerations

While the preceding cost-benefit analysis was in accordance with current practices for economic analysis of seismic risk mitigation, there are some important factors that have been neglected. This section identifies these factors and discusses their potential impacts.

5.9.3.1 The Effects of Risk Averseness

The analyses herein have been based on expected losses (or benefits) and costs and the assumption of a linear relationship between money and utility. That is, there is no preference between, say, a 100% chance of losing \$100 and a 1% chance of losing \$10,000 (both of which have an expected cost of \$100). However, for low-probability, high-loss situations, this may not be entirely appropriate because the consequences may be different. The most likely situation would be for stakeholders to be “risk averse.” In this situation, the relationship between money and utility is non-linear. Figure 5.25 shows example utility functions of varying levels of risk aversion, with dollars, x , (millions) as the abscissa and utility, $U(x)$, as the ordinate.

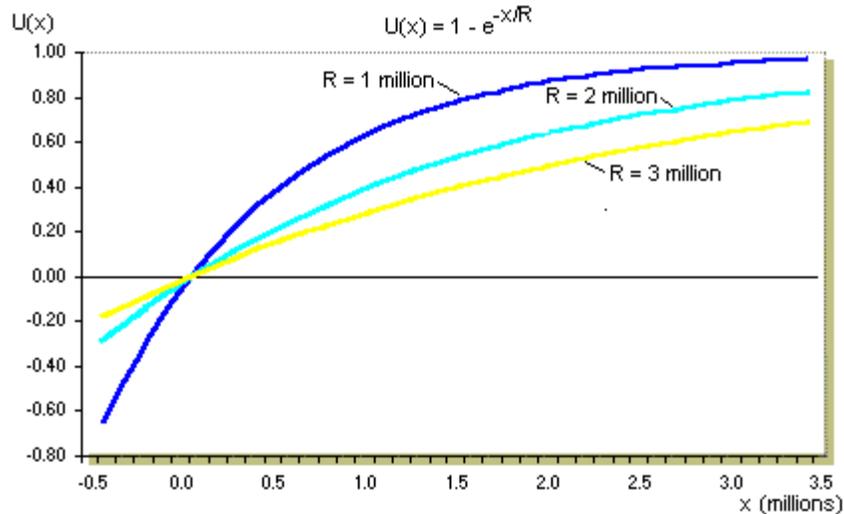


Figure 5.25 – Example Risk-Averse Utility Functions

(From Lee Merkhoffer Consulting, 2014)

Take, for example, the blue curve (*note: $R=1$ million is a parameter defining the curve*). A project with a 50% chance of $x=+1.0$ million and a 50% chance of $X=-0.5$ million. The expected value of this project would be $x=+0.25$ million. However, in terms of utility, the project has a 50% chance of $U(x)=+0.6$ and a 50% chance of $U(x)=-0.7$. The expected value of the *utility* is -0.05 . Thus, from an expected cost point of view, one would conclude this is a good investment, but from an expected utility point of view it is not.

Smith and Vignaux (2006) showed that accounting for risk-averseness could result in about a 10-100% increase in net present value of future losses (for a hypothetical “typical building”), over a reasonable range of risk-averseness. While we have not accounted for this in our analysis, it is perhaps one argument for supporting strengthening measures that have a BCR of less than unity.

5.9.3.2 The Effects of Insurance

In the cost-benefit analysis, it was assumed that the owners would suffer the losses due to building repair and for tenant relocation and lost rental income. However, owners may choose to purchase earthquake insurance. In this case, owners would only be responsible for building repairs up to a certain deductible (likely 10 or 15% of the building value). As aforementioned, there are many factors contributing to the level of loss an owner experiences and insurance is one such factor.

Since the quantification of building damage is directly in terms of the building value (in the form of mean damage factors), one can readily account for insurance in the model: rather than summing the entire damage costs over the range of shaking, the costs are summed up to a mean damage factor equal to the deductible. Assuming a deductible of 10% and recalculating, we find that the expected costs (to the owner) due to building damage are fairly consistent at about 65-75% of the value for the uninsured case. This is because the majority of the risk is contained in the low-intensity (high probability) region. Of course, the effects on the overall cost-benefit ratio are even less pronounced because it has no effect on the remaining types of losses. Figure 5.26 provides an example calculation directly from the model for braced-parapet buildings (note: the BCR's are from a previous model, and are not up to date, but the conclusions remain the same).

Adjusted		Uninsured		Insured	
Sa(1)	%/50 Yrs	MDF(Sa(1))	Ann. MDF	MDF(Sa(1))	Ann. MDF
0.01	99.9%	0.01%	0.0008%	0.01%	0.0008%
0.02	98.4%	0.07%	0.0035%	0.07%	0.0035%
0.06	79.3%	0.83%	0.0092%	0.83%	0.0092%
0.08	64.1%	1.41%	0.0115%	1.41%	0.0115%
0.11	46.1%	3.45%	0.0290%	3.45%	0.0290%
0.22	18.0%	10.84%	0.0233%	10.00%	0.0215%
0.34	8.7%	16.55%	0.0140%	10.00%	0.0085%
0.45	4.8%	23.36%	0.0094%	10.00%	0.0040%
0.56	2.8%	32.47%	0.0055%	10.00%	0.0017%
0.67	2.0%	37.80%	0.0064%	10.00%	0.0017%
0.78	1.2%	42.59%	0.0022%	10.00%	0.0005%
0.89	0.9%	46.92%	0.0024%	10.00%	0.0005%
1.01	0.7%	52.66%	0.0051%	10.00%	0.0005%
1.12	0.4%	56.05%	0.0015%	10.00%	0.0002%
1.23	0.3%	59.15%	0.0015%	10.00%	0.0002%
1.34	0.3%	61.98%	0.0015%	10.00%	0.0002%
1.45	0.2%	65.80%	0.0007%	10.00%	0.0001%
1.56	0.1%	68.09%	0.0007%	10.00%	0.0001%
1.68	0.1%	70.20%	0.0003%	10.00%	0.0000%
1.79	0.1%	72.16%	0.0003%	10.00%	0.0000%
1.90	0.1%	73.97%	0.0003%	10.00%	0.0000%
2.01	0.1%	76.43%	0.0003%	10.00%	0.0000%
2.12	0.1%	77.93%	0.0003%	10.00%	0.0000%
2.23	0.0%	79.32%	0.0000%	10.00%	0.0000%
		Σ Ann. MDF=	0.12950%	Σ Ann. MDF=	0.09355%
		Ann. Cost=	\$2,694	Ann. Cost=	\$1,946
		50-Yr Cost=	\$134,679	50-Yr Cost=	\$97,295
		PV=	\$49,174	PV=	\$35,524
		Savings=	\$10,761	Savings=	\$6,628
		Total Savings=	\$37,769	Total Savings=	\$33,636
		Upgrade Cost=	\$24,000	Upgrade Cost=	\$24,000
		B/C Ratio=	1.57	B/C Ratio=	1.40

Figure 5.26 – Effect of Insurance for Braced-Parapet Buildings (Site Class C)

With regards to benefit-cost considerations, the “public” (i.e. life-safety) benefits are obviously unaffected by insurance. Of course, the owner BCR could be increased or decreased over the uninsured case depending on the cost of the insurance. The important conclusion is that insurance (under the parameters assumed herein) eliminates only about 25% of the annual expected losses.

5.10 Sensitivity Analyses

The foregoing results were based on several assumptions. This section presents sensitivity analyses for some of the key parameters as listed below:

- Building Replacement Value
- Cost of Upgrades
- Discount Rates
- Time Horizon
- Structural Vulnerability
- Length of Street Front Exposure
- Value of a Statistical Life

5.10.1 Building Replacement Value

In our base case, we assumed a building replacement value of \$260/sq.ft. This was the URM replacement value used by Thibert (2008), which was based on research specific to buildings in British Columbia. However, some studies (Rutherford & Chekene 1990) have suggested that, since one cannot replace a URM building (due to code restrictions), that the replacement value of a similar reinforced concrete building should be used. Thibert specified a value of \$300/sq.ft. and so this served as our ‘high’ value. HAZUS (FEMA 2012) meanwhile specifies replacement values based on occupancy. The total building value derived from HAZUS for our building was \$225/sq.ft. and this served as our ‘low’ value. Table 5.27 provides the resulting overall BCR’s for the high and low values. Refer to Table 5.25 for the base case values.

Table 5.27 – Sensitivity Results for Building Replacement Value

Site Class	Overall Benefit/Cost Ratio					
	Braced Parapet		Partial Retrofit		Full Retrofit	
	High	Low	High	Low	High	Low
B	0.83	0.75	0.35	0.31	0.14	0.12
C	1.44	1.31	0.63	0.56	0.25	0.22
D	2.12	1.95	0.93	0.82	0.38	0.33
E	4.18	3.89	1.78	1.61	0.73	0.64

As can be seen, the results are not overly sensitive to the building replacement value. This is because the reduced building damage losses only represent about 25-50% of the benefits.

5.10.2 Cost of Upgrades

The costs of upgrades are highly variable for actual projects, so their sensitivity is of interest. The low and high estimates were defined in Section 5.8. Table 5.28 provides the resulting overall BCR's. Refer to Table 5.25 for the base case values.

Table 5.28 – Sensitivity Results for Costs of Upgrades

Site Class	Overall Benefit/Cost Ratio					
	Braced Parapet		Partial Retrofit		Full Retrofit	
	High	Low	High	Low	High	Low
B	0.40	1.19	0.23	0.55	0.09	0.25
C	0.69	2.06	0.42	0.99	0.16	0.46
D	1.01	3.04	0.62	1.46	0.23	0.68
E	2.01	6.04	1.21	2.82	0.45	1.33

The shown above results are highly sensitive to the costs of upgrades, as cost is the denominator in the benefit/cost ratio. Noticeable changes in conclusions appear in the braced parapet category where upgrading is only viable for buildings on soft soils for the higher cost. The increased cost of \$6/sq.ft. is likely applicable for some buildings (eg. building footprint shapes with a great deal of perimeter or smaller buildings).

5.10.3 Discount Rates

The discount rates were one of the most difficult parameters to define for this study, as it involves both tangible factors (eg. rate of return for riskless investment) and intangible ones (consumer time preference, intergenerational considerations). The high and low discount rates were defined in Section 5.5.8.2. Table 5.29 provides the resulting BCR's of our sensitivity analysis.

Table 5.29 – Sensitivity Results for Discount Rates

Site Class	Overall Benefit/Cost Ratio					
	Braced Parapet		Partial Retrofit		Full Retrofit	
	High	Low	High	Low	High	Low
B	0.58	1.17	0.24	0.48	0.09	0.19
C	1.00	2.03	0.43	0.87	0.17	0.34
D	1.48	3.00	0.64	1.28	0.26	0.51
E	2.92	5.98	1.23	2.50	0.50	1.00

5.10.4 Time Horizon

The time horizon for our base case was 50 years. The appropriate time horizon seems to be a point of contention since the Seattle study (Gibson Economics 2014) used a base case of 30 years, while a study in New Zealand (Martin Jenkins Associates 2012) used a base case of 75 years. Here, we use 75 years as our upper bound, and 25 years as our lower bound. Table 5.30 presents the results of our sensitivity analysis for time horizon.

Table 5.30 – Sensitivity Results for Time Horizon

Site Class	Overall Benefit/Cost Ratio					
	Braced Parapet		Partial Retrofit		Full Retrofit	
	High	Low	High	Low	High	Low
B	0.88	0.57	0.36	0.24	0.14	0.09
C	1.53	0.98	0.66	0.43	0.26	0.17
D	2.27	1.45	0.97	0.63	0.39	0.26
E	4.53	2.85	1.89	1.21	0.76	0.50

The results are not very sensitive to increases in time horizon; this is because the discount rates curtail the added benefits. For the low end, the changes are noticeable, but they do not represent any changed conclusions regarding the viability of upgrades.

5.10.5 Structural Vulnerability

In Chapter 4 we saw that the vulnerability of buildings from different regions was highly variable. As such, it was decided to include alternate structural vulnerabilities in our analyses. Section 4.9.3 shows the alternate structural vulnerabilities. No alternative non-structural vulnerabilities were tested, as there is a relative paucity of data in the literature; however, these are also important and warrant further future investigation. Table 5.31 provides the results of our sensitivity analysis.

Table 5.31 – Sensitivity Results for Structural Vulnerability

Site Class	Overall Benefit/Cost Ratio					
	Braced Parapet		Partial Retrofit		Full Retrofit	
	High	Low	High	Low	High	Low
B	0.94	0.58	0.37	0.26	0.14	0.11
C	1.65	1.08	0.68	0.50	0.26	0.21
D	2.45	1.57	1.00	0.74	0.39	0.31
E	5.00	3.03	1.99	1.39	0.77	0.59

The results are only modestly sensitive to structural vulnerability. This is because much of the value of the building lies in the non-structural components and contents.

5.10.6 Length of Street Front Exposure

The length of streetfront exposure is a parameter that will vary highly from building to building, both because of location within a block (middle, end, isolated) and location within a community (eg. a URM building set back on industrial land without public access). The base case used a streetfront exposure of 30 lineal feet, representing a typical dimension for a row building. The upper bound was defined as 90 feet and the lower bound was defined as zero feet (i.e. no pedestrian exposure whatsoever). Table 5.32 presents the results of our sensitivity analysis for streetfront exposure.

Table 5.32 – Sensitivity Results for Streetfront Exposure

Site Class	Overall Benefit/Cost Ratio					
	Braced Parapet		Partial Retrofit		Full Retrofit	
	High	Low	High	Low	High	Low
B	1.24	0.57	0.48	0.25	0.18	0.10
C	2.44	1.08	0.97	0.50	0.37	0.21
D	3.72	1.55	1.47	0.72	0.56	0.30
E	7.44	3.08	2.91	1.38	1.12	0.58

The pedestrian exposure has a significant impact, although very few conclusions regarding the viability of upgrades are changed. Nonetheless, the pedestrian (and occupant) exposure are very important parameters in the potential losses and should be considered when deciding to upgrade buildings.

5.10.7 Value of a Statistical Life

The value of a statistical life (VSL) is again a difficult but necessary parameter in most cost-benefit analyses. Recommended values as recently published by the U.S. Department of Transportation (USDOT 2013) were used in this study; high and low value values for sensitivity analyses are explicitly defined in this document and they are presented in Section 5.5.6.4. Table 5.33 presents the results of our sensitivity analysis.

Table 5.33 – Sensitivity Results for VSL

Site Class	Overall Benefit/Cost Ratio					
	Braced Parapet		Partial Retrofit		Full Retrofit	
	High	Low	High	Low	High	Low
B	0.99	0.59	0.40	0.26	0.15	0.10
C	1.71	1.03	0.71	0.47	0.28	0.19
D	2.55	1.50	1.06	0.68	0.41	0.29
E	5.16	2.87	2.11	1.26	0.83	0.54

The sensitivity to VSL is fairly similar to most of the other parameters in that the difference is noticeable, but most of the conclusions are not changed.

5.11 Limitations of Cost-Benefit Analysis & Expected Costs

Through our cost-benefit analysis, we have focused on expected values of costs and benefits. Although this is the most common metric used in risk assessment, it is not the only one. Indeed, some have argued that decision-making based solely on expected cost can be fallacious, as it directly equates high-probability/low-consequence losses with low-probability/high-consequence losses (Haines 2004), yet decision-makers are generally not risk-neutral. Haines recommends that decisions for rare events be based on what is called the “Partitioned Multi-objective Risk Method”, which essentially separates the expected values of catastrophic events as a conditional probability from the overall expected value. Each loss value then becomes a separate objective function in a multi-objective optimization scheme.

Smith (2003) used Monte Carlo simulation to derive the entire distribution of benefits for an earthquake strengthened building. The results are shown in Figure 5.27 (from the original paper). Note that there is over a 50% chance that approximately zero benefits (shown here in thousands of dollars) will be accrued throughout the life of the building. The maximum benefit (not shown) was \$382,000, while the expected value was \$13,000. Smith argues that, because the distribution of the benefits is highly skewed, characterizing the benefits purely by its expected value is inadequate.

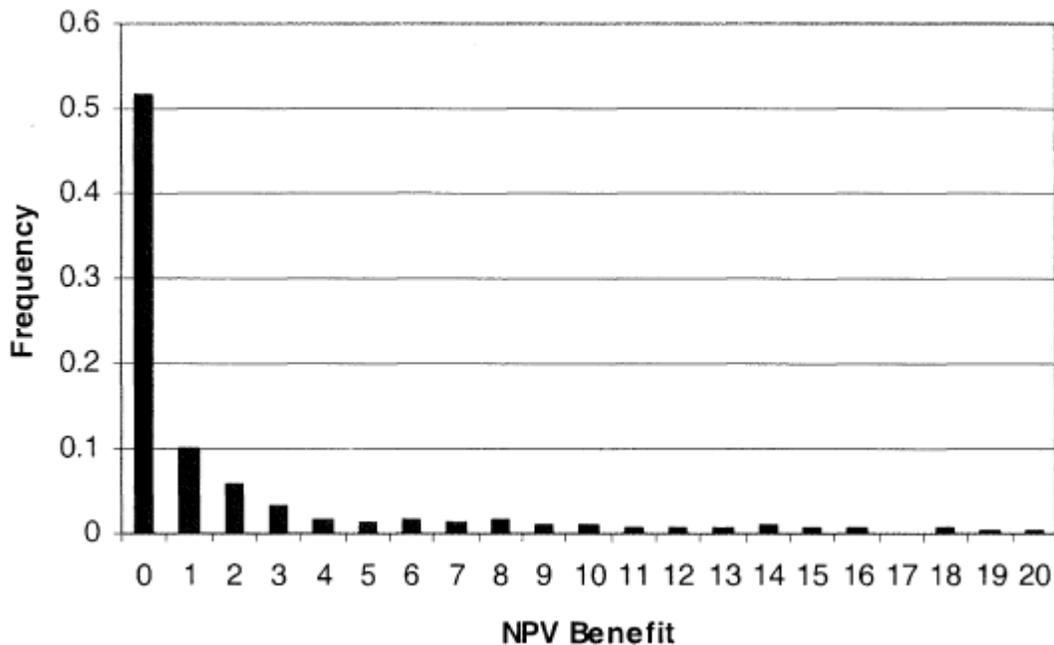


Figure 5.27 – Distribution of Benefits from Strengthening

(From Smith 2003)

Finally, it should be noted that many infrastructure projects are not justified purely from a cost-benefit point of view – examples in Victoria include the replacement of the Johnson Street Bridge or the proposed construction of a new sewage treatment plant. Such projects are undertaken because society has identified minimum levels of safety, transportation service, or environmental responsibility that must be adhered to. Thus, to make earthquake strengthening decisions solely based on expected cost would put seismic risk mitigation at a disadvantage relative to other efforts.

5.12 Summary

The purpose of this chapter was to investigate the economic validity of potential seismic risk mitigation measures for unreinforced masonry buildings, specifically commercial occupancy buildings in downtown Victoria. To this end, the expected costs and benefits were quantified for various degrees of retrofitting to a “typical” building.

The benefits lie in the reduced expected losses due to earthquakes. The losses for each level of strengthening were estimated using a methodology generally consistent with HAZUS (FEMA 2012). However, a number of refinements were made, as noted below:

- Structural damage was based on statistics collected from URM buildings in real earthquakes, as presented in Chapter 4

- Damage to NSC's & Contents were based on HAZUS, but with an adjustment to account for collapse effects
- Downtime estimates were based on HAZUS with adjustments to account for 'scale effects'; the adjustments were subjective, but were based on experience in Christchurch
- Fatality estimates were based on HAZUS (for occupants) and Rutherford and Chekene (1990) for pedestrians; hospitalized injuries were taken as four times the number of casualties, again based on the work of Rutherford and Chekene
- Casualties were monetized based on VSL guidance in a recent study by the United States Department of Transportation (USDOT 2013)

The cost of seismic retrofitting was quantified for each level:

- Retrofit costs were based on detailed estimates by the author and/or data from actual seismic retrofitting projects in Victoria
- The costs included allowances for consulting fees, taxes, and other soft costs

The benefits were converted to present value and compared with the costs:

- Separate discount rates were applied to the owner and public benefits; the base rates were 5% and 3%, respectively; 50 years was the base time horizon
- Parapet upgrades were found to have a BCR ≥ 1 for almost all buildings in the base case (site class B had a BCR=0.79)
- Partial retrofits (taken here to mean only tension ties) were found to have a BCR ≥ 1 on very soft soils (site class E)
- Full retrofits did not achieve a BCR of 1 under the base case; the BCR was 0.68 for site class E
- Owner-only BCR's (i.e. excluding casualties) were calculated and generally did not exceed unity. Casualty reduction generally accounted for 40-60% of the benefits
- Parapet upgrades were found to achieve the majority of the expected casualty reductions (about 70% of that for a full retrofit); damage reduction, however, was more modest

Sensitivity analyses were conducted:

- The highest sensitivity parameters were cost of upgrades and the discount rates
- Length of streetfront exposure was modestly sensitive, but in real buildings it is expected to be highly variable

Other effects such as risk aversion, and insurance were briefly investigated. Limitations of expected costs were discussed, as were additional decision-making criteria.

5.13 Conclusions

5.13.1 General Conclusions

The cost-benefit analysis showed that parapet upgrades were good to excellent investments for the vast majority of buildings. For buildings on soft soils, partial retrofits appear to be good investments. The BCR for full retrofits exceeded 1 in only the most favourable scenarios in our sensitivity analysis. However, this cannot be taken as conclusive evidence that full retrofits are a bad investment. In any case, it must be remembered that these results are not building specific and are highly variable and uncertain. They are merely one of many possible indicators for decision-making.

Of course, a significant portion of the benefits lie in reduced casualties; the public accrues these benefits, rather than the building owner. This is potentially a basis for cost-sharing of seismic retrofits.

Based on the earlier discussions, it is clear that the expected values of benefits should not be the sole consideration in earthquake strengthening. Recall also that some effects (such as risk-averseness) and some benefits (such as preservation of heritage) were not considered in the cost-benefit analysis. Overall, it is felt that a BCR should serve as a first pass indicator, in that any result above 1.0 should be conclusive, while BCR's above about 0.8 to 1.0 are possible candidates, depending on the level of risk averseness and the value placed on heritage preservation. For the remainder of cases, it would be recommended to establish multi-objective models for decision-makers. For example, we may seek to minimize overall expected costs (due to retrofitting and earthquake damage), while constraining our results to a certain maximum number of fatalities for a 475-year event or to have a certain fraction of heritage buildings not be demolished.

5.13.2 Conclusions For Victoria

As discussed in Chapter 3, it appears that seismic risk mitigation efforts in Victoria are lacking in comparison to other regions in the Pacific Northwest (and further abroad), despite some success in the Tax Incentive and Building Incentive Programs.

Based on the results of this chapter, it appears that an incentive program for parapet bracing or partial retrofitting is a good use of public money. As demonstrated herein, parapet bracing achieves a significant portion (approximately 70%) of the expected

casualty reduction of a full retrofit. This is because it is highly effective at reducing casualties in the low shaking intensity (high probability of occurrence) region of the hazard curve. Of course, the benefits eventually disappear at higher shaking intensities, but such events are rare.

Based on the cost-benefit results, a few different incentive schemes could be justified: simply providing parapet bracing for all buildings would be one option. Another would be to pursue partial retrofits for buildings on soft soils. Or perhaps all buildings could be eligible for partial retrofits, considering the heritage preservation value not included in this analysis. Finally, one might consider providing partial retrofits for more vulnerable buildings (eg. 3+ storey buildings, isolated buildings). Of course, these are just a few examples based strictly on the results herein, and there are likely to be a number of other factors to be considered (eg. would a building owner accept that their neighbor is eligible for a larger grant?)

A last item worth pointing out is that *earthquake insurance is not a substitute for strengthening*, or vice-versa – especially for partial measures. In fact, partial retrofit measures such as parapet bracing and tension ties are quite complementary to insurance in that they address the low-intensity/high-probability earthquakes, while insurance serves to cover the more calamitous (but rare) events.

5.14 Future Research Opportunities

In preparing the cost-benefit analysis, two facts became abundantly clear: firstly, lost estimating/risk assessment is a highly multi-disciplinary field that should ideally have input from many different professions. While the opportunities for input from other professions were limited during the preparation of this thesis, an effort was made to clearly document each component and decision so that future researchers and/or decision-makers can readily accept, reject, or refine the premises.

Secondly, a considerable amount of judgement was required to make up for a paucity of data in many areas and there are a number of components that could be improved. Some good candidates for future research are as briefly discussed below.

5.14.1 Damage to Nonstructural Components

The nonstructural (and contents) vulnerabilities in HAZUS are only refined by “code level,” not by building type. Given that the dynamics of flexible diaphragm URM buildings are substantially different from most modern buildings, one might expect the

damage patterns to be different. For example, one might expect to see more damage to floor and ceiling finishes and less damage to partitions and windows.

Furthermore, the effect of collapse/heavy structural damage on the non-structural losses was accounted for in a simplistic, but rational manner. Validating this concept with observed damage statistics would be valuable.

5.14.2 Downtime Estimates

Experience from the Christchurch earthquake showed that the downtime of a given building is also highly dependent upon the damage to the overall building stock. This may be due to safety concerns or a lack of supply in construction. Comerio (2006) showed similar experiences in California earthquakes, albeit on a smaller scale. Of course there are likely several other factors including the legal/political environments and the community's general level of experience in dealing with earthquakes; having never suffered a damaging earthquake, Victoria will likely struggle in this regard.

5.14.3 Improved Casualty Estimation

Casualties are clearly an important component of the risk assessment. Unfortunately, detailed information on human exposure was not available. Occupant and pedestrian densities were based on the values specified by Rutherford and Chekene (1990), with some subjective reductions to account for the smaller population of Victoria, BC compared to San Francisco, CA.

A detailed study on pedestrian densities in the downtown area, accounting for spatial, temporal and seasonal variation would likely be valuable for Victoria in many aspects even beyond seismic risk assessments.

5.14.4 Complete Database of URM Buildings in Victoria

In the San Francisco study, Rutherford and Chekene made use of a complete database of the URM buildings to produce loss estimates. A similar database would be extremely valuable for assessing seismic risk in Victoria. Such information might include the number and location of buildings, levels of strengthening, number of stories, occupancies, lineal footage of streetfront exposure, and many more items.

5.14.5 Scenario Loss Estimates

Consideration was given to providing loss estimates and cost-benefit analyses for scenario earthquakes for Victoria. However, it was ultimately decided that such an effort would

imply a higher level of certainty than could be achieved at this time. Scenario estimates are highly tangible and (perhaps unfortunately) are more likely to spur on policymaking than the benefit-cost analysis completed herein. It is recommended that scenario estimates be pursued after refining various input parameters with Victoria-specific data.

5.14.6 Improved Risk Assessment

As discussed herein, expected costs/benefits alone are not necessarily a sufficient metric for risk assessment, as expected costs fail to capture risk averseness and the use of expected costs alone is not consistent with typical decision-making for other infrastructure projects. Utility theory and multi-objective risk assessment methods are the next step in identifying risk mitigation measures worth pursuing.

In general, the risk assessment performed herein (including cost benefit analyses) should be considered indicative, or preliminary. A methodology has been developed, but several more pieces of information specific to Victoria (as discussed above) should be collected prior to results being used as input to decision-making on seismic safety and upgrading.

Chapter 6

Methodologies for Assessing and Prioritizing URM Seismic Risk

6.1 Purpose and Scope

In Chapter 3 it was shown that URM seismic risk mitigation efforts were generally lacking in Victoria (and southwestern BC in general) compared to other regions in the Pacific Northwest that are subjected to similar hazards. In the two preceding chapters, it was shown that there were systematic differences in the seismic performance of various URM buildings and that it is generally a worthwhile investment to perform some form of strengthening.

When dealing with a large set of buildings and a finite amount of resources, it is important to be able to identify which buildings should receive strengthening first. Since identification and screening is essentially always the first step in URM seismic risk mitigation, it is important that an appropriate methodology be established. The purpose of this chapter is to identify and adapt screening methodologies suitable for use with URM buildings in Victoria and other locations in southwestern BC. To this end, we review a host of methods that were encountered, recommend existing methodologies for Victoria and make improvements to these methodologies as deemed necessary.

6.2 Existing Methodologies

There is a modestly long history of assessing and prioritizing seismic risk in North America and abroad. There are several methodologies; some have been widely used, while others have not. Some have been based on sound engineering and rationale, while others have not. Some methodologies have been developed for application nation-wide, while others been ad hoc solutions developed by cities as part of seismic strengthening ordinances. This section reviews several methodologies, with a particular focus on their application to URM.

6.2.1 California URM Prioritization Methods

When the state of California adopted Senate Bill 547 in 1986 (California Legislature 1986), also known as the “URM Law,” communities were obligated to identify URM

buildings within their jurisdiction and establish some form of loss reduction program (as discussed in Chapter 3). Many jurisdictions wound up mandating earthquake strengthening and, as such, had to prioritize buildings for funding, and assign compliance timelines. Meanwhile, other communities such as Los Angeles had already implemented seismic strengthening ordinances.

6.2.1.1 Los Angeles

The City of Los Angeles’ URM strengthening ordinance, Division 88 (City of Los Angeles 1985), involves a “rating classification” that affects timelines for compliance and design force levels. The rating classification is essentially based on building function occupant load, as shown in Table 6.1.

Table 6.1 – Los Angeles Prioritization

Rating Classification	Definition	Full Compliance Deadline (w/ partial retrofit)
Essential Building (Highest Priority)	Medical/Emergency Services Centers	3 years
High-Risk Building	>100 Occupants	3.25 years
Medium-Risk Building	All Others	4-6 years (depending on occupant load)
Low-Risk Building (Lowest Priority)	<20 Occupants	7 years

As expected, higher priority buildings had shorter timelines for compliance: essential buildings were to be strengthened within three years of their owners being served notice from the City that their building fell within the scope of the ordinance. Owners of lower priority buildings had the option of extending the deadline for full compliance by performing partial retrofits (parapet bracing, plus tension anchors). Design forces also were increased for "high-risk" and "essential" buildings 33% and 86%, respectively.

6.2.1.2 San Francisco

The city of San Francisco provides another example of an ad hoc prioritization system as part of a retrofit ordinance. Buildings were assigned “Risk Levels.” Table 6.2 shows the definitions and compliance deadlines (which are measured from February 1993). Note that Group A occupancies are Assembly types, and Group E occupancies are Educational occupancies, which includes only grade schools (through grade 12) and day cares.

Table 6.2 – San Francisco Prioritization

Risk Level	Definition	Compliance Deadline
Level 1 Buildings (Highest Priority)	Group A occ. (300+ occupants) Group E occupancies 4+ storey buildings on poor soil	3.5 years
Level 2 Buildings	Non-Level 1 buildings located on poor soils (in certain high-density areas, such as downtown)	5 years
Level 3 Buildings	Non-Level 1 buildings located on poor soil in other areas (i.e. lower density)	11 years
Level 4 Buildings (Lowest Priority)	All other URM buildings	13 years

The risk level also affected strengthening design requirements. Some non-level 1 buildings that also met a variety of other requirements could achieve compliance with a partial retrofit, known as “Bolts-Plus” (see Chapter 3).

This prioritization takes into account a few more factors than the Los Angeles system. It explicitly accounts for poor soils, which was shown to be a significant risk factor in Chapter 5; it also indirectly accounts for the pedestrian exposure by discriminating amongst the locations within the city. It does not account for benefits of partial strengthening (as did the Los Angeles system), but the timelines for compliance are generally longer.

6.2.1.3 Sonoma County

Sonoma County devised a points system to prioritize URM buildings for its seismic retrofit ordinance. The points system accounted for the following factors:

- Occupant load (0 to 5 points)
- Number of stories (1 to 3 points)
- Proximity to public sidewalks (0 to 1 points)
- Proximity to adjacent buildings (0 to 1 points)
- Partial retrofits (-0 to -3 points)

Increasing points correspond to increasing risk. Based on the results, buildings were categorized as low, moderate, or high priorities, with final compliance deadlines as noted in Table 6.3.

Table 6.3 – Sonoma County Prioritization

Risk Level	Definition	Compliance Deadline
High Priority	> 6 points	4.5 years
Moderate Priority	4 – 6 points	7 years
Low Priority	< 4 points	12 years

Note that partial retrofitting measures could, at most, reduce a buildings priority by one level. Additionally, one important factor not included here is soils.

6.2.2 Seattle

More recently, the City of Seattle has begun developing a policy on URM earthquake strengthening. In developing its prioritization system and timelines for compliance, a thorough review of the California methods was undertaken (City of Seattle 2014). Ultimately, a prioritization system was devised as shown in Table 6.4.

Table 6.4 – Seattle Prioritization

Risk Level	Definition	Compliance Deadline
Critical	Schools Essential facilities	7 years
High	3+ storeys on poor soil, or >100 occupants (for assembly occupancy)	10 years
Medium	All other URM buildings	13 years

This prioritization system accounts for most of the important risk factors, although pedestrian exposure is a noticeable omission.

6.2.3 FEMA 154

In contrast to the URM-specific, ad hoc prioritization systems described above, more general “seismic screening” methodologies have been developed by various national agencies. FEMA 154 (FEMA 2002a), entitled “Rapid Visual Screening of Buildings for Potential Seismic Hazards”, is one such example.

The first edition of the document was published in 1988 and the second edition was published in 2002. Here, we will focus on the second edition; a discussion on the first versus second editions is provided in the second edition of FEMA 155 (FEMA 2002b), which is a technical companion document. During the writing of this thesis, a new (third) edition of FEMA 154 was in development, but had not yet been published. As such, the work herein focuses exclusively on the second edition. FEMA 154 is quite general in that it addresses many building types and characteristics, including:

- Seismic force resisting type (eg. steel moment frame, concrete shear wall)
- Number of storeys
- Structural irregularities (eg. weak storey, torsional sensitivity)
- Era of construction
- Seismicity of the region (low, medium, high)
- Site soils

The methodology is a points-like system, whereby a structure of a given seismic force resisting type is associated with a “Basic Structural Hazard” (BSH) score and then the score is modified by adding or subtracting score modifiers based on the remaining characteristics. The BSH equals the negative logarithm of the probability of collapse for the building under MCE (maximum considered earthquake) demands, which for the United States is taken as 2/3 of the soils-adjusted spectral acceleration, with a probability of exceedance of 2% in 50 years (FEMA 2002b). The equation for the BSH is as shown below:

$$\text{BSH} = -\log_{10}[\text{P}(\text{collapse} \mid \text{MCE})] \quad (6-1)$$

Where:

BSH \equiv Basic Structural Hazard score

P(collapse | MCE) \equiv Probability of collapse, given MCE demands

MCE \equiv 2/3* F_a * S_s or 2/3* F_v * S_1 (F_a, F_v, S_s, S_1 as defined in ASCE 7-10)

And the final score is computed by applying score modifiers, as per the equation below:

$$S = BSH \pm \Sigma SM \quad (6-2)$$

Where:

BSH \equiv Basic Structural Hazard score

SM \equiv Score Modifiers

In general, the BSH's and score modifiers were derived from HAZUS99 fragilities (NIBS 1999), which are based on the capacity spectrum method of analysis. Score modifiers were derived either by using appropriate different existing fragilities (eg. low/medium/high-rise), code-specified modification factors (eg. soil types), or by judgmental increases in the ground motion intensity (eg. vertical irregularities).

Figure 6.1 shows the screening form for “High Seismicity” ($S_s \geq 0.5g$, $S_1 \geq 0.2g$) locations, including the BSH's and SM's for all buildings. FEMA 154 specifies a typical “cut-off” score of 2.0: according to FEMA 154, buildings with scores below this threshold should be subject to a seismic evaluation by a design professional, although it is noted that user's should decide on an appropriate cut-off score for their specific project.

Since the methodology is based on collapse, its primary concern is life safety. The fact that it does not incorporate occupant/pedestrian exposure or the presence of fall hazards into the scoring is a shortcoming. As can be seen, however, the data collection sheets do provide space to record this information. There is also limited guidance on the effects of seismic upgrading: FEMA 154 (FEMA 2002a) suggests classifying upgraded URM according to the type of new lateral system that was introduced (eg. if braced frames were provided, classify the structure as S1, see Figure 6.1). However, partial retrofits, and even some full retrofits, will not introduce such elements, so this solution has limited usefulness.

6.2.4 NRC Screening Methodology

A Canadian methodology for seismic screening of buildings was also developed shortly after the first edition was FEMA 154. The National Research Council of Canada's “Manual for Screening of Buildings for Seismic Investigation” was published in 1993 (NRC 1993). We will henceforth refer to this methodology as ‘NRC93’.

Rapid Visual Screening of Buildings for Potential Seismic Hazards
FEMA-154 Data Collection Form

HIGH Seismicity

	<p>Address: _____ _____ Zip _____</p> <p>Other Identifiers _____</p> <p>No. Stories _____ Year Built _____</p> <p>Screener _____ Date _____</p> <p>Total Floor Area (sq. ft.) _____</p> <p>Building Name _____</p> <p>Use _____</p> <div style="text-align: center; margin-top: 50px;">PHOTOGRAPH</div>														
Scale: _____															
OCCUPANCY	SOIL	TYPE	FALLING HAZARDS												
Assembly Commercial Emer. Services	Govt Historic Industrial	Office Residential School	Number of Persons 0 – 10 11 – 100 101-1000 1000+	A Hard Rock B Avg. Rock C Dense Soil D Stiff Soil E Soft Soil F Poor Soil	<input type="checkbox"/> Unreinforced Chimneys <input type="checkbox"/> Parapets <input type="checkbox"/> Cladding <input type="checkbox"/> Other: _____										
BASIC SCORE, MODIFIERS, AND FINAL SCORE, S															
BUILDING TYPE	W1	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM
Basic Score	4.4	3.8	2.8	3.0	3.2	2.8	2.0	2.5	2.8	1.6	2.6	2.4	2.8	2.8	1.8
Mid Rise (4 to 7 stories)	N/A	N/A	+0.2	+0.4	N/A	+0.4	+0.4	+0.4	+0.4	+0.2	N/A	+0.2	+0.4	+0.4	0.0
High Rise (> 7 stories)	N/A	N/A	+0.6	+0.8	N/A	+0.8	+0.8	+0.6	+0.8	+0.3	N/A	+0.4	N/A	+0.6	N/A
Vertical Irregularity	-2.5	-2.0	-1.0	-1.5	N/A	-1.0	-1.0	-1.5	-1.0	-1.0	N/A	-1.0	-1.0	-1.0	-1.0
Plan irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Pre-Code	0.0	-1.0	-1.0	-0.8	-0.6	-0.8	-0.2	-1.2	-1.0	-0.2	-0.8	-0.8	-1.0	-0.8	-0.2
Post-Benchmark	+2.4	+2.4	+1.4	+1.4	N/A	+1.6	N/A	+1.4	+2.4	N/A	+2.4	N/A	+2.8	+2.6	N/A
Soil Type C	0.0	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4
Soil Type D	0.0	-0.8	-0.6	-0.6	-0.6	-0.6	-0.4	-0.6	-0.6	-0.4	-0.6	-0.6	-0.6	-0.6	-0.6
Soil Type E	0.0	-0.8	-1.2	-1.2	-1.0	-1.2	-0.8	-1.2	-0.8	-0.8	-0.4	-1.2	-0.4	-0.6	-0.8
FINAL SCORE, S															
COMMENTS														Detailed Evaluation Required	
														YES	NO

* = Estimated, subjective, or unreliable data
DNK = Do Not Know

BR = Braced frame
FD = Flexible diaphragm
LM = Light metal

MRF = Moment-resisting frame
RC = Reinforced concrete
RD = Rigid diaphragm

SW = Shear wall
TU = Tilt up
URM INF = Unreinforced masonry infill

Figure 6.1 – High Seismicity Screening Form for FEMA 154, 2nd Edition
(From: FEMA 154, 2002)

NRC93 was reportedly based on the first edition of FEMA 154 (NRC 1993). However, the scoring system is fundamentally different: instead of being based on collapse probabilities, it is based on a comparison of code-specified earthquake forces between the 1990 National Building Code of Canada (NBCC), and the NBCC version that was in effect at the time of the subject building's construction. More specifically, the following factors are accounted for:

- Index A: Seismicity
- Index B: Soil Conditions
- Index C: Type of Structure (eg. steel braced frame, concrete shear wall)
- Index D: Structural Irregularities
- Index E: Building Importance
- Index F: Nonstructural Hazards (life-safety hazards, vital operation hazards)

The scoring system calculates a Structural Index (SI) and a Nonstructural Index (NSI). The SI and NSI are calculated as shown below:

$$SI = A*B*C*D*E \quad (6-3)$$

$$NSI = B*E*F \quad (6-4)$$

Where:

A ≡ Seismicity Index

B ≡ Soils Index

C ≡ Structure Type Index

D ≡ Irregularities Index

E ≡ Importance Index

F ≡ Nonstructural Index

These indices are then added together to provide the final score known as the Seismic Priority Index (SPI), as shown below:

$$SPI = SI + NSI \quad (6-5)$$

Where:

SI ≡ Structural Index

NSI ≡ Nonstructural Index

The scoring system is somewhat more detailed than FEMA 154. This is primarily because NRC93 was designed to include interior access and thus items such as occupancy and non-structural components can be incorporated into the scoring; another

factor is that the treatment of irregularities is more detailed: the NRC93 irregularities index was taken from the first edition was FEMA 154, which included specific items such as soft storeys, and torsional sensitivities. In the second edition of FEMA 154, irregularities were simplified to just “vertical irregularity” or “plan irregularity” because more refinements could not be accounted for analytically. Figure 6.2 shows the NRC93 screening form, including all the indices.

As previously mentioned, the various indices are derived based on comparisons of code provisions from NBCC 1990 and the code in effect at the time of construction. Taking soils conditions as an example, we can see that a pre-1965 building located on “soft soil” received $B=1.5$, which is equal to the appropriate design foundation factor, ‘F’ from NBCC 1990; a post-1965 building, however, is assigned $B=1.0$, because the effect of soils is presumed to have been included in the design. This raises an interesting question about the FEMA 154 methodology and the appropriateness of applying score modifiers for soils conditions regardless of the era of construction.

The main merit of NRC93 versus FEMA 154 is that the former offers more “resolution” in the results. This is particularly relevant when dealing with just one type of building, such as URM buildings: the FEMA system will result in many repeated scores, since the only difference between buildings will often be soils.

The main drawback of NRC93 versus FEMA 154 is that it is less scientifically justified. The FEMA system represents a clear metric of seismic risk – the probability of collapse and hence serious life-safety threats. The NRC93 system is less clear: many of the indices are based on judgment or code-specified factors which are often not entirely objective either. Documentation on its development is also scarce. As a result of NRC93’s lack of a clear scientific basis and associated documentation, it is likely less legally defensible than FEMA 154, which is important for communities wishing to identify privately-owned hazardous buildings.

SEISMIC SCREENING FORM											p. 2 of 2		ITEM No.:																									
SEISMIC PRIORITY INDEX: Circle appropriate value and enter each result on right side. Use asterisk (*) with uncertain values																																						
A	Seismicity	Design NBC	Effective Seismic Zone (Z_v , or $Z_v + 1$ if $Z_a > Z_v$)										A =																									
			2	3	4	5	6																															
		Pre - 65	1.0	1.5	2.0	3.0	4.0																															
		65 - 84	1.0	1.0	1.3	1.5	2.0																															
		Post - 85	1.0	1.0	1.0	1.0	1.0	1.0																														
B	Soil Conditions	Design NBC	Soil Category										B =																									
			Rock or Stiff Soil	Stiff Soil > 50 m	Soft Soil > 15 m	Very Soft or Liquefiable Soil	Unknown Soil																															
		Pre - 65	1.0	1.3	1.5	2.0	1.5																															
		Post - 65	1.0	1.0	1.0	1.5	1.5																															
C	Type of Structure (BM = Benchmark year, see p.1)	Design NBC	Construction Type and Symbol (see p. 1)												C =																							
			Wood		Steel			Concrete		Precast		Masonry Infill		Masonry																								
		WLF	WPB	SLF	SMF	SBF	SCW	CMF	CSW	PCF	PCW	SIW, CIW	RML, RMC	URM																								
		Pre - 70	1.2	2.0	1.0	1.2	1.5	2.0	2.5	2.0	2.5	2.0	3.0	2.5		3.5																						
		70 - BM	1.2	2.0	1.0	1.2	1.5	1.5	1.5	1.5	1.8	1.5	2.0	1.5	3.5																							
		Post - BM	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	—																								
D	Building Irregularities	Design NBC	1. Vertical	2. Horiz.	3. Short Concrete Columns	4. Soft Storey	5. Pounding	6. Modification	7. Deterioration	8. None	D = product of circled Numbers (Max of 4.0) =																											
			Pre - 70	1.3	1.5	1.5	2.0	1.3	1.3	1.3		1.0																										
		Post - 70	1.3	1.5	1.5	1.5	1.3	1.0	1.3	1.0																												
E	Building Importance	Design NBC	Low Occupancy N < 10	Normal Occupancy N = 10 - 300	School, or High Occupancy N = 301 - 3000	Post Disaster, or Very High Occup. N > 3000	Special Operational Requirements	E =																														
			Pre - 70	0.7	1.0	1.5	2.0		3.0																													
		Post - 70	0.7	1.0	1.2	1.5	2.0																															
<p>$N = \text{Occupied Area} \times \text{Occupancy Density} \times \text{Duration Factor}^* = \dots \times \dots \times \dots =$</p> <table border="0"> <tr> <td>Primary Use:</td> <td>Occupancy Density Persons / m²</td> <td>Average Weekly Hours of Human Occupancy</td> <td>* Duration Factor is equal to the average weekly hours of human occupancy divided by 100, not greater than 1.0</td> </tr> <tr> <td>Assembly</td> <td>1</td> <td>5 - 50</td> <td></td> </tr> <tr> <td>Mercantile, Personal service</td> <td>0.2</td> <td>50 - 80</td> <td></td> </tr> <tr> <td>Offices, Institutional, Manufacturing</td> <td>0.1</td> <td>50 - 60</td> <td></td> </tr> <tr> <td>Residential</td> <td>0.05</td> <td>100</td> <td></td> </tr> <tr> <td>Storage</td> <td>0.01 - 0.02</td> <td>100</td> <td></td> </tr> </table>															Primary Use:	Occupancy Density Persons / m ²	Average Weekly Hours of Human Occupancy	* Duration Factor is equal to the average weekly hours of human occupancy divided by 100, not greater than 1.0	Assembly	1	5 - 50		Mercantile, Personal service	0.2	50 - 80		Offices, Institutional, Manufacturing	0.1	50 - 60		Residential	0.05	100		Storage	0.01 - 0.02	100	
Primary Use:	Occupancy Density Persons / m ²	Average Weekly Hours of Human Occupancy	* Duration Factor is equal to the average weekly hours of human occupancy divided by 100, not greater than 1.0																																			
Assembly	1	5 - 50																																				
Mercantile, Personal service	0.2	50 - 80																																				
Offices, Institutional, Manufacturing	0.1	50 - 60																																				
Residential	0.05	100																																				
Storage	0.01 - 0.02	100																																				
SI	STRUCTURAL INDEX = A · B · C · D · E =										SI =																											
F	NON - STRUCTURAL HAZARDS		Description (see p. 1)			None	Yes	Yes *	F = max (F ₁ , F ₂)																													
	F ₁	Falling Hazards to Life			Pre - 70 NBC	1.0	3.0	6.0																														
					Post - 70 NBC	1.0	2.0	3.0																														
	F ₂	Hazards to Vital Operations			Any Year	1.0	3.0	6.0																														
* applies only if one or more of the following descriptors on page 1 are circled: SMF, CMF, soft storey, torsion																																						
NSI	NON - STRUCTURAL INDEX = B · E · F =										NSI =																											
SPI	SEISMIC PRIORITY INDEX = SI + NSI =										SPI =																											
Comments:																																						

Figure 6.2 –Screening Form From NRC93 (2nd of 2 pages)
(From: NRC, 1993)

One final drawback of NRC93 is that it is associated with a version of the NBCC that is now quite outdated. However, researchers at the University of Ottawa have recently updated the methodology (Saatcioglu, Shooshtari and Foo 2013) to be compatible with the 2010 NBCC. This included revising the indices A, B, and C; the remaining indices were unchanged from the original NRC93 values. The updated methodology has been incorporated into a software package known as “SCREEN” (Saatcioglu, Shooshtari and Foo 2013). One word of caution with regards to URM is that the program SCREEN applies short period foundation factors: on soft soils, this actually reduces the design base shear, giving more favourable results for buildings on site class E. However, we have seen throughout this study that URM buildings are often not short period buildings and our damage data has consistently shown increased damage for buildings on soft soils.

6.2.5 New Zealand IEP

As discussed in Chapter 3, New Zealand has national legislation requiring territorial authorities to adopt policies on “earthquake-prone” buildings (NZSEE 2006). Of course, identification of buildings is the first major step in any seismic risk mitigation strategy. The New Zealand Society for Earthquake Engineering has provided guidelines for screening of buildings in the form of an “Initial Evaluation Procedure” (IEP) (NZSEE 2006). Figure 6.3 shows a flow chart for the IEP.

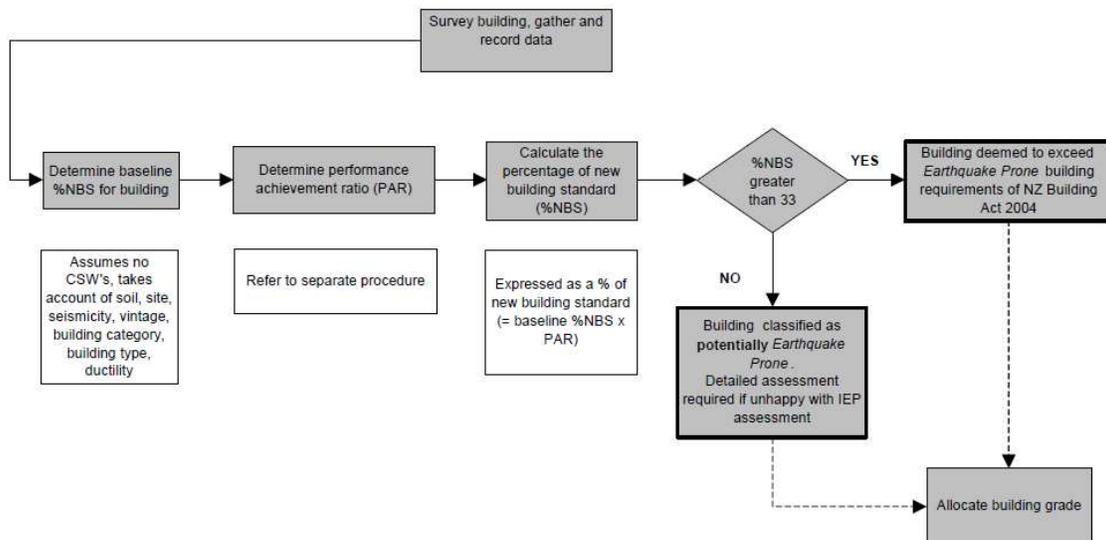


Figure 6.3 – New Zealand IEP Procedure Flow Chart
(From: NZSEE, 2006)

Like FEMA 154 and NRC93, the IEP methodology was developed as a general prioritization system covering all possible types of buildings. Similar to NRC93, the IEP is based on a comparison of code-specified design forces. However, it is somewhat more

explicit in this sense, because the results are expressed in %NBS (New Building Standard). It covers a similar host of issues as NRC93, including era of construction, soils, structure type, irregularities, and importance. It does not directly include the occupant load/density in the calculation of the %NBS, but it does provide guidance on including it when prioritizing amongst buildings. Some interesting factors accounted for in the IEP that are not included in NRC93 are “Near-Fault” and a “Site Characteristics” (eg. landslide, liquefaction) factors.

As previously noted, URM has several potential vulnerabilities that are not addressed by general screening methodologies. To address this, the NZSEE (2006) guidelines contain an appendix with an alternate IEP for URM buildings (i.e. either method can be used). It is a detailed points system, as shown in Figure 6.4, below.

Item		Attribute ranking				Assessed score	
		0	1	2	3	Long	Trans
1	Structure continuity	Excellent	Good	Fair	Poor or none		
2	Configuration						
2a	Horizontal regularity	Excellent	Good	Fair	Poor		
2b	Vertical regularity	Excellent	Good	Fair	Poor		
2c	Plan regularity	Excellent	Good	Fair	Poor		
3	Condition of structure						
3a	Materials	Sound	Good	Fair	Poor		
3b	Cracking or movement	Not evident	Minor	Moderate	Severe		
4	Wall (URM) proportions						
4a	Out of plane	Good			Poor		
4b	In-plane	Excellent	Good	Fair	Poor		
5	Diaphragms						
5a	Coverage	Excellent	Good	Fair	Poor		
5b	Shape	Excellent	Good	Fair	Poor		
5c	Openings	None			Significant		
6	Engineered connections between floor/roof diaphragms and walls, and walls and diaphragms capable of spanning between	Yes			No		
7	Foundations	Excellent	Good	Fair	Poor		
8	Separation from neighbouring buildings	Adequate			Inadequate		
Total Attribute Score:::				<i>for each direction</i>			
				<i>for building as a whole:</i>			

Figure 6.4 – New Zealand Alternate IEP for URM Buildings

(From: NZSEE, 2006)

Essentially, the building is assigned 0 to 3 points for each attribute and the points are summed, with higher points indicating worse performance. The %NBS is then calculated based on 1) the total number of points and 2) the results of certain items (from Figure 6.4).

Table 6.5 – New Zealand URM IEP Points vs. %NBS

Level	Attribute Scores	%NBS
1	0 for all attributes	67
2	≤ 1 for items #1-6; ≤ 2 for items #7-8	35
3	0 for item #1; ≤ 1 for items #2-6; ≤ 2 for items #7-8	40
4	$5 < \text{total points} \leq 10$	20
5	$10 < \text{total points} \leq 15$	15
6	$15 < \text{total points} \leq 25$	10
7	total points > 25	5

This system is clearly the most detailed prioritization system of those reviewed. Unfortunately, there is little documentation on its development. One notable omission is the effect of soils, which has been shown to be a highly important factor.

Ultimately, the primary purpose of the IEP is to identify buildings that are *potentially* earthquake-prone ($< 34\%$ NBS) for further, more detailed, assessment. Based on a review by the author, it appears that a significant amount of partial strengthening would be needed to reach the 34%NBS threshold. For a “typical” unstrengthened building (as we have maintained throughout this study), a score upwards of 25 is rather easily justified. In order to achieve Level 2 (35%NBS), the following measures would likely be required:

- Improved continuity (Item #1): provide reinforced bands at all floors, or alternate gravity support systems for all URM walls
- Improved configuration (Item #2): reduced eccentricity (perhaps through wood cross walls)
- Repointing of weak mortar (note: it is generally not recommended to repoint masonry with mortars having differing stiffness)
- Out-of-plane bracing for walls exceeding certain slenderness ratios
- Anchors at all floors/roof

It appears that the procedure is designed to conclude that virtually all URM buildings that have not been fully strengthened (i.e. for in-plane and out-of-plane demands) are potentially earthquake-prone, which is a sound conclusion for the purposes of screening.

6.2.6 Summary of Existing Procedures

Several methodologies for assessing and/or prioritizing URM seismic risk have been reviewed herein. Every method seemed to have its merits and at least one notable flaw. Depending on the intent of the evaluation, certain methods may be more appropriate than others. Many of the simple ad hoc methods developed by cities or counties have the benefit that they are easily understood by the public and easily administered by the building officials. The main merit of FEMA 154 is that it is based on a clear metric of performance (the probability of collapse, and hence serious life-safety threats); this makes it also somewhat easy to communicate to the public and may make it more legally defensible. Unfortunately, it does not contain any provisions for strengthened URM buildings (or the various specific deficiencies that may be present). The NRC93 method has the benefit of being more detailed in the number of risk factors it considers, but the results are more difficult to interpret. Finally, the New Zealand IEP is likely the most detailed. However, it also suffers from a lack of documentation and scientific basis (particularly for the "alternate" points system for URM), similar to the NRC93 method.

6.3 Improvements to Methodology

Throughout the course of this study, the various screening methods were applied to a sampling of buildings in downtown Victoria. In doing so, it became clear that improvements to one or more existing methodologies were warranted, particularly for the case where the goal is to identify hazardous buildings (as opposed to prioritizing for strengthening): this is because our methodology needs to be sufficiently detailed so as be able to distinguish amongst the buildings and yet must also be legally defensible, as concerns regarding liability often arise when identifying hazardous buildings.

6.3.1 Selected Methodology for Adaptation/Improvement

Ideally, adapting/improving the NRC93 methodology for our particular purpose would have been the first choice, as it is a Canadian methodology. However, its lack of documentation and scientific basis was not something that could be overcome. Similar conclusions were reached regarding the New Zealand IEP. As such, it was decided that adapting/improving the FEMA 154 methodology was the best option.

6.3.2 Improvements

Based on our results from Chapter 5, the following items were identified as important seismic risk factors that could be included in an improved model.

- The effects of strengthening
- Occupant/pedestrian exposure
- Number of storeys

All of the above can be accounted for by either:

- 1) modifying the probability of collapse (based on our new structural fragilities) and recalculating the BSH
- 2) multiplying the BSH by a factor to achieve an equivalent risk of fatality (eg. a building with double the fatality rate should have half the probability of collapse)

6.3.2.1 The Effect of Strengthening

The original FEMA 154 BSH scores were derived by calculating the probability of collapse based on the fragilities and collapse rates contained in HAZUS99 (NIBS 1999). Because new HAZUS-compatible fragilities were derived for each level of strengthening, these can be directly used to calculate new BSH's for each level of strengthening.

Essentially, one has only to extract the value for the fraction of buildings in the “complete” damage state at the appropriate level of spectral acceleration, which is then multiplied by the conditional probability of collapse, for buildings in the complete damage state, as specified in HAZUS. FEMA 155 specifies that the probability of collapse be calculated at the MCE demands: for the United States, this is two-thirds of spectral acceleration having a probability of exceedance of 2% in 50 years. We will maintain this philosophy here and take 2/3 of the appropriate spectral acceleration, assuming Site Class B, consistent with the U.S. The appropriate value for Victoria is $S_a(1)=0.54g$ (see the Hazard Curve in Appendix D); the final value is calculated as shown below:

$$S_a(1)_{MCE} = 2/3 * S_a(1)_{2\%/50yr, B} = 2/3 * 0.54g = \underline{0.36g} \quad (6-6)$$

Where:

$S_a(1)_{MCE} \equiv S_a(1)$ for use in calculating the fraction of buildings in a complete damage state

$S_a(1)_{2\%/50yr, B} \equiv S_a(1)$ for site class B having a 2% in 50 years probability of exceedance

From our base case fragility curves (see Appendix C), the corresponding fraction of buildings expected to be in the “complete” damage state was obtained and subsequently

the conditional probability of collapse, as specified in FEMA 155 (FEMA 2002b), was used to calculate the probability of collapse. Table 6.6 shows the results.

Table 6.6 – Calculated BSH by Strengthening Level

Parameter	Unret.	Parapets Braced	Partial Retrofit	Full Retrofit
P(DS=Complete MCE)	1.1%	0.2%	0.12%	0.04%
P(Collapse DS=Complete)	0.17%	0.030%	0.018%	0.006%
BSH Score	2.78	3.52	3.75	4.22

The BSH scores are noticeably higher than those given in FEMA 154. This is primarily because our observed damage statistics (upon which our fragilities are based) showed less damage than the HAZUS curves at this level of shaking (see Figure 4.40, for example).

Because the desire is for our new BSH scores to be compatible with FEMA 154, they will be calibrated to be consistent with those in the document. The BSH scores for the “Unretrofitted” case will be set equal to the value from FEMA 154. For the “Full Retrofit” case, the BSH will be set equal to the value equal to 2.8, which is the value specified for RM1 (flexible diaphragm, reinforced masonry) buildings; the rationale for this is that multiple documents have suggested that reinforced masonry is likely the closest match for a typical strengthened URM building (Rutherford & Chekene 1997, FEMA 2012). The two intermediate values will be scaled between these two values so that the final *relative* values are the same as for the calculated values. Table 6.7 shows the final BSH scores for each level of retrofitting. Notice that an additional strengthening level, “Partial Parapets,” has been included: this represents buildings that have only braced a portion of their parapets (i.e. likely the largest ones, overlooking the busiest areas). Although there is no statistical basis specifically for partial bracing, it was felt important to recognize the life-safety benefits. Based on the foregoing results, the following score modifiers to account for retrofitting are proposed.

Table 6.7 – Final Score Modifiers for Strengthening

Parameter	Unret.	Partial Parapets	Parapets Braced	Partial Retrofit	Full Retrofit
BSH Score	1.8	2.0	2.4	2.5	2.8
Score Modifier	--	+0.2	+0.6	+0.7	+1.0

6.3.2.2 Occupant/Pedestrian Exposure

Presumably the results of the FEMA 154 methodology were intended to represent an average of many buildings. As such, we will assume that the FEMA-specified BSH accounts for an average level of pedestrian exposure and we will adjust the BSH up or down to account for higher and lower occupant/pedestrian exposure. The question remains whether the overall occupant load, or the occupant density is the appropriate parameter: using the overall occupant load would put certain buildings at a disadvantage simply due to their size; although at first one may surmise that the increased number of occupants means increased risk, we must remember that the goal is to identify hazardous buildings or prioritize amongst buildings. In considering many buildings and a finite supply of resources, there is no reason to believe that retrofit money would be better spent on fewer, larger buildings, as opposed to many smaller ones. As such, it was decided to use occupant/pedestrian *densities*.

Based on the occupant densities provided by Rutherford and Chekene (1990), it can be seen that the time averaged occupant density across all occupancy types is about 2 occupants per 1000 sq.ft. The highest is commercial at about 3.6 and the lowest is industrial at about 1.1. As a rather crude approximation, we will take this as double and half the average density. By doubling and halving the probability of collapse and recalculating the BSH, we find score modifiers as shown in Table 6.8 for differing levels of occupancy. Finally, intermediate values (Low/Moderate and Moderate/High) are also provided to recognize mixed-use type occupancies.

Table 6.8 – Score Modifiers for Occupant Density

Parameter	Low Density (eg. industrial)	Low/ Moderate	Moderate (eg. residential)	Moderate/ High	High Density (eg. comm'l)
Score Modifier	+0.3	+0.15	--	-0.15	-0.3

Some consideration was given to specifying a score modifier for high-importance or post-disaster type facilities, but such an adjustment would have been arbitrary, since the building vulnerability and life-safety risk are not inherently different. Instead, it is suggested that an alternate cut-off score be used. FEMA 155 notes that scores of 2.5 and 3 have been used for critical and other higher-performance facilities (FEMA 2002b).

As demonstrated in this study and elsewhere (Ingham and Griffith 2011b), it is generally more dangerous to be immediately outside a URM building than it is to be inside. As

such, we should also account for the pedestrian density. Rutherford and Chekene (1990) provided pedestrian densities for San Francisco, depending on location within the city. The busiest downtown areas have a time-averaged density of approximately 65persons/1000ft of sidewalk, while other areas away from the downtown core have time-averaged densities of 30person/1000ft and 11persons/1000ft. Here, we can again approximate this as twice/half the central value of 30persons/1000ft; this results in the same modification factors, which are provided in Table 6.9.

Table 6.9 – Score Modifiers for Pedestrian Density

Parameter	Low Density	Typical	High Density
Score Modifier	+0.3	--	-0.3

Of course, the aforementioned pedestrian densities were for a highly urbanized city. Smaller cities with less pedestrian activity could perhaps justify classifying their busiest areas as ‘Typical’ and the remainder as ‘Low Density.’ However, this adjustment is likely of little interest where the goal is simply to establish relative levels of risk between buildings. For Victoria, we will maintain our assumption that the busiest areas of the downtown core are classified as High-Density areas – the locations will be formally delineated in Chapter 7.

6.3.2.3 Building Typology

The final item to account for is the building typology. As shown in Figures 4.21 and 4.29, certain building typologies as developed by Russell and Ingham (2010) were found to have differences in their vulnerability for both the Loma Prieta and Canterbury earthquakes. Recall from Table 4.22 that we were able to define 3 categories of vulnerability:

- Lower Vulnerability: Types A and B (i.e. 1-storey)
- Moderate Vulnerability: Types D and F (2 and 3-storey row buildings)
- Higher Vulnerability: Types C, E, G (2 and 3-storey isolated and industrial/religious buildings)

The MDF adjustment factors for $S_a(1)=0.3g$ in Table 4.22 will be used, as this approximates Victoria’s MCE demands: this gives a $\pm 25\%$ increase in damage. Although these statistics are not directly in terms of collapse, we will assume a corresponding increase and decrease. Multiplying our probabilities of collapse by 0.75 and 1.25, and recalculating our BSH, we can derive the score modifiers as shown in Table 6.10. Finally, it was decided to recognize the increased vulnerability of end

buildings (Ingham and Griffith 2011b) and, as such, Type D and F “end of row” buildings are classified as Higher Vulnerability.

Table 6.10 – Score Modifiers for Building Typology

Parameter	Lower Vulnerability (Types A, B)	Moderate Vulnerability (Types D, F)	Higher Vulnerability (Types C, E, G)
Score Modifier	+0.1	--	-0.1

Since retrofits would presumably correct many of the deficiencies, *these adjustment factors will not be applied to buildings with partial or full retrofits* (i.e. they only apply to unretrofitted or braced-parapet buildings).

While other, more specific characteristics have been correlated with damage in some studies, the results have often been mixed, and it is difficult to deal with the effects of multiple characteristics because their presence may be correlated with each other and their effects likely are not superimposeable. This minor adjustment based on typology appears to be the most consistent with observed damage statistics (as demonstrated in Chapter 4) and serves the desired purpose of providing more ‘resolution’ in our screening results.

6.4 Summary

The purpose of this chapter was to identify and adapt screening methodologies suitable for use with URM buildings in Victoria and other locations in southwestern BC. To this end, the following was undertaken:

- Several existing ad-hoc methods for prioritizing seismic retrofitting were reviewed
- Three general methods for seismic screening were reviewed: FEMA 154, NRC93, and the New Zealand IEP
- The FEMA 154 methodology was identified as the best candidate to be adapted to the particular needs of this study
- Improvements were made to FEMA 154 to render it more suitable for URM-only surveys
- Improvements included accounted for retrofit status, occupant/pedestrian exposure, and building typology

6.5 Conclusions

6.5.1 General Conclusions

Our review of the ad-hoc prioritization methods showed significant variation in detail and in the related consequences (i.e. retrofit deadlines). As expected, a good prioritization method must be quite simple, so as to be easily understood by the public and implemented by city staff; yet, it should also account for all the important aspects of risk. This includes the seismic hazard (implicit in the community's decision to adopt a retrofit ordinance), building vulnerability (eg. number of storeys), soils, building function (eg. essential facilities), and occupant/pedestrian exposure. In the author's opinion, the San Francisco prioritization system is the best balance between accuracy and simplicity, while the Seattle system is a close second, because it does not account for soils, presumably due to a lack of sufficient publicly available information on soil conditions.

Our review of the general seismic screening methods showed that each of the three methods had flaws – at least with respect to our particular goal of completing a URM-only screening project. Because the FEMA 154 method was the most readily adaptable and scientifically justified, it was selected as the best candidate for improvements. These improvements will help overcome its lack of 'resolution' as previously discussed. These modifications could be implemented for URM buildings in any high seismicity city on the west coast of North America, provided the buildings conform to the general form (as discussed in Chapter 2) and typologies as discussed in Section 4.8.2.3. For areas of lower seismicity, the score modifiers should be recalculated.

6.5.2 Conclusions for Victoria

The purpose of this chapter was to identify/adapt appropriate methodologies for URM screening in Victoria (and elsewhere in southwestern BC).

In terms of a method for prioritizing buildings as part of a retrofit ordinance, it is recommended that either the San Francisco method or Seattle method be adopted. The question is essentially whether or not the building authorities wish to include soils information in the prioritization. Soil conditions for seismic design have been developed for Victoria (BC Ministry of Energy and Mines 2000). However, the maps indicate that they are only suitable for regional assessment, and not for local assessment (i.e. on average they are accurate, but there may be local errors due to interpolations in the mapping).

In terms of a general methodology (eg. for identifying hazardous URM buildings throughout a community), it is recommended that the FEMA 154 methodology be used, along with the modifications presented herein. At this point, it is recommended that the modifications not be used if the survey is not exclusive to URM buildings, as no effort has yet been made to ensure compatibility.

6.6 Future Research Opportunities

While the brief review included herein indicated that either the San Francisco or Seattle methods would be best for prioritizing URM retrofits under an ordinance, there are several other factors that could come into play, most of which fall outside the scope of engineering. A more high-level review on the success/failings of these systems would be warranted before making a decision as to which method is most suitable for Victoria.

In terms of the general methodologies, perhaps the most interesting question is whether or not the metric employed FEMA 154 is the most appropriate for its potential uses. In the author's opinion, the probability of collapse (modified to account for occupant/pedestrian exposure) is a good indicator of life-safety risk, but it is only for one particular seismic demand, towards the lower probability end of the spectrum. A method based on the full extent of the hazard curve may be more scientifically justified. Another issue is that the methodology deals essentially only with life-safety. The appropriateness of the method for screening projects focusing on reducing expected costs of damage (say, governments considering seismic rehabilitations) is a topic that deserves further investigation.

Finally, during the writing of this thesis, a third edition of FEMA 154 was under development. However, the document had not yet been published and so the efforts here focused solely developing new score modifiers compatible with the second edition framework. Further review should be completed, subsequent to the release of the third edition.

Chapter 7

Inventory & Screening of Victoria's URM Buildings

7.1 Purpose and Scope

In Chapter 3, it was seen that URM seismic risk mitigation efforts in Victoria were lacking in comparison to other regions. It was also seen that, while risk mitigation programs varied greatly by jurisdiction, a publicly available inventory of buildings was always a key first step. Inventories allow the magnitude of the problem to be quantified, raise public awareness (and thus pressure on building owners), facilitate enforcement, and allow progress to be tracked.

The purpose of this chapter is not to develop an inventory for Victoria fit for the aforementioned uses. As noted in Chapter 3, it is recommended that the City of Victoria commission an inventory as part of an overall URM seismic risk mitigation program. Rather, the intent is to summarize the available data and present potential methodologies for future implementation. Additionally, the existing available data will be used to make a preliminary characterization of the building stock.

7.2 Data Collection

Two sources of data were used in this study. They are noted below and discussed in the subsequent sections.

- 1) A survey of over 300 buildings, completed by a City of Victoria building official in 1989 (City of Victoria 1989)
- 2) A more targeted survey of approximately 80 buildings in 2013, completed as part of this study

7.2.1 Existing Survey

In 1989, a City of Victoria building official, Murdock McDonald, conducted a survey of 329 buildings in the downtown core of Victoria. The survey extents were somewhat similar to present-day “Old Town.” Figure 7.1 shows a map from the original document, indicating the extents of the survey.

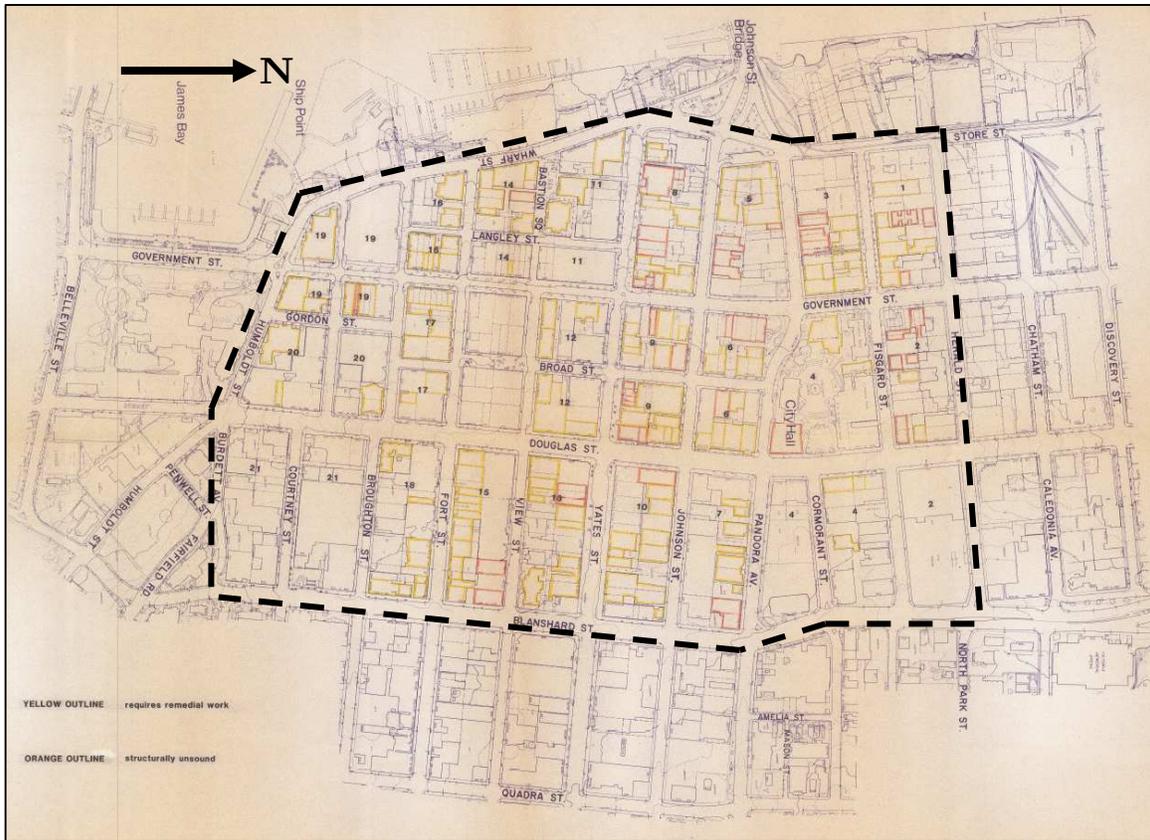


Figure 7.1 – 1989 Survey Extents

(Modified From: City of Victoria, 1989)

The survey involved interior access to many of the buildings included items as follows:

- Civic address
- Earliest plumbing date (proxy for construction date)
- Type of construction (URM, reinforced concrete, steel, wood frame)
- Occupancy
- Useable floor area
- Rooftop structures (i.e. presence of fall hazards)
- Number of storeys

Particular attention was paid to fall hazards – the condition, height, location on the building and areas below threatened by the fall hazards were recorded. Also recorded was an assessment of each building’s seismic resistance: buildings were assigned to one of four categories, with buildings in “Category 1” being considered as meeting building code standards of the day and buildings in “Category 4” as having *“virtually no structural seismic resistance.”*

Although the survey is quite dated, it is still relatively useful in that it contains a complete inventory of the buildings within those boundaries at the time of the survey. Since it is a complete sample, we can establish the prevalence of URM construction relative to other types in the area. Of the 329 buildings surveyed, 260 (79%) were identified as being of URM construction. Figure 7.2 below shows the breakdown for all buildings. The buildings are classified into general material types, including unreinforced masonry (URM), reinforced concrete (RC), reinforced masonry (RM), steel frame (SF), and wood frame (WF).

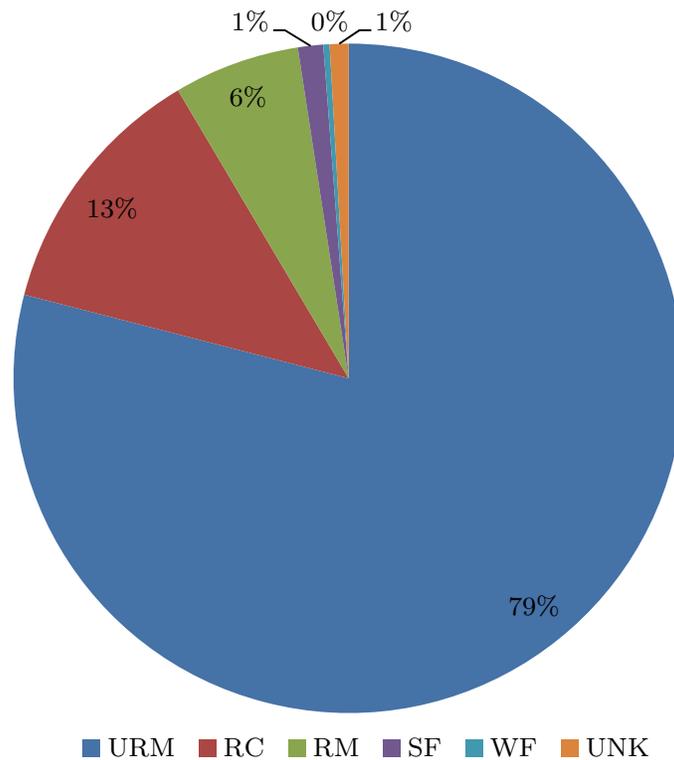


Figure 7.2 – Breakdown of Buildings by Construction Type (329 Buildings)

In looking specifically at the URM buildings, the first interesting breakdown is by date of construction (as indicated by the “earliest plumbing date” field). Unfortunately, the records prior to the 20th century are limited and the earliest record is 1892, despite the fact that many buildings were constructed prior to this date. As such it was decided to group any pre-1900 buildings. Figure 7.3 provides the breakdown for 254 of the buildings (6 did not have a recorded date). As shown in the figure, about half of the buildings were constructed before 1900, with numbers sharply declining after 1920. The latest recorded date was 1962.

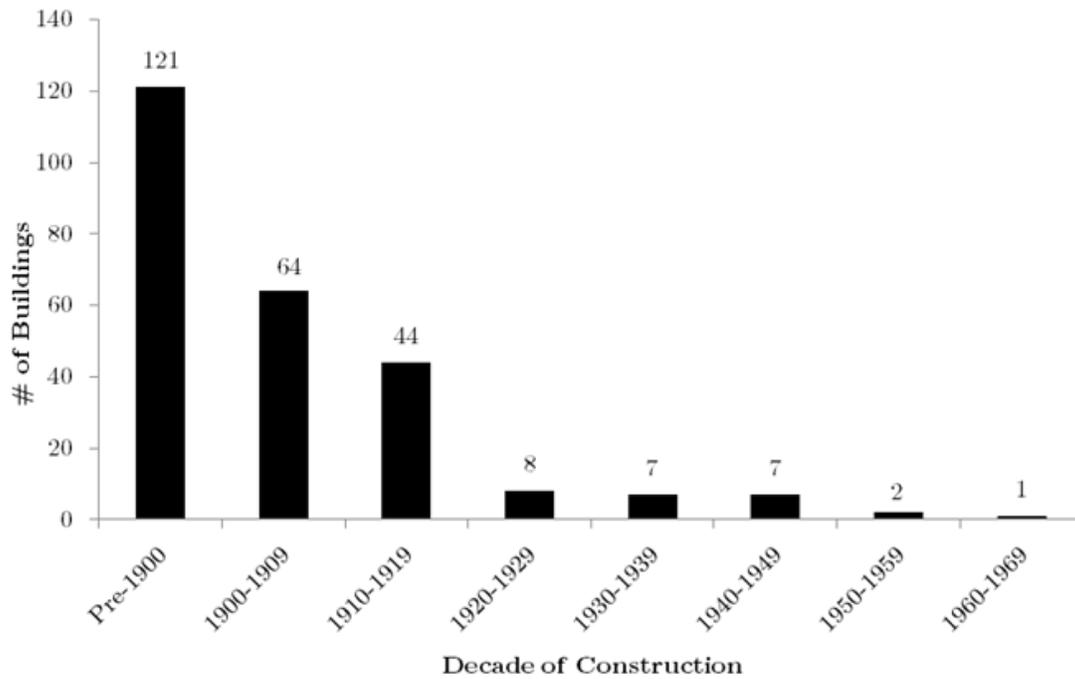


Figure 7.3 – Victoria URM Buildings by Decade of Construction (254 Buildings)

Another interesting breakdown is by number of storeys. Figure 7.4 shows the breakdown for 258 URM buildings (2 did not have an entry).

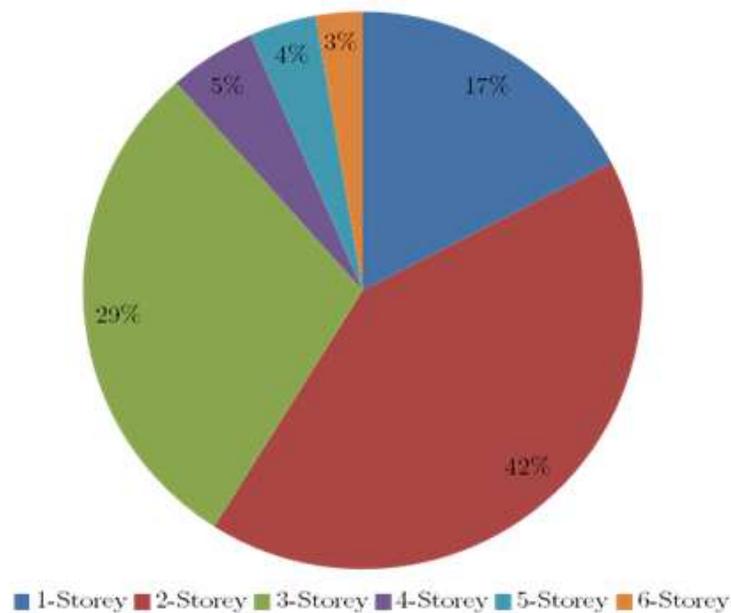


Figure 7.4 – Victoria URM Buildings by Number of Storeys (258 Buildings)

The statistics show that 1, 2, and 3 storey buildings comprise the majority (88%) of the URM building stock. However, there are also appreciable numbers of taller buildings, with the tallest being six storeys. It should be noted that this is likely not representative for outlying areas, where one to three storey buildings are presumed to be even more prevalent (in proportion to taller buildings).

7.2.2 Pilot Survey of Buildings

While the 1989 survey provided some interesting overall statistics, it was obviously outdated. Additionally, it did not contain sufficient information for application of the chosen screening assessment (see Chapter 6). As the purpose of this portion of the work was to develop a screening methodology and provide example results, it was decided that a new “pilot survey” would be completed for a specific subset of buildings in Victoria’s Old Town.

The pilot survey included 81 (unreinforced masonry) buildings within the area shown in Figure 7.5. This area was identified by the industry sponsor as the target area and was selected primarily for three reasons:

- 1) Buildings in this area are almost exclusively URM
- 2) It was known that there was a mix of strengthened and unstrengthened buildings
- 3) This area encompasses the major arterial routes to the Johnson street bridge; failure of these buildings (and resulting debris in the streets) would, in addition to posing a fall hazard, potentially hamper recovery efforts

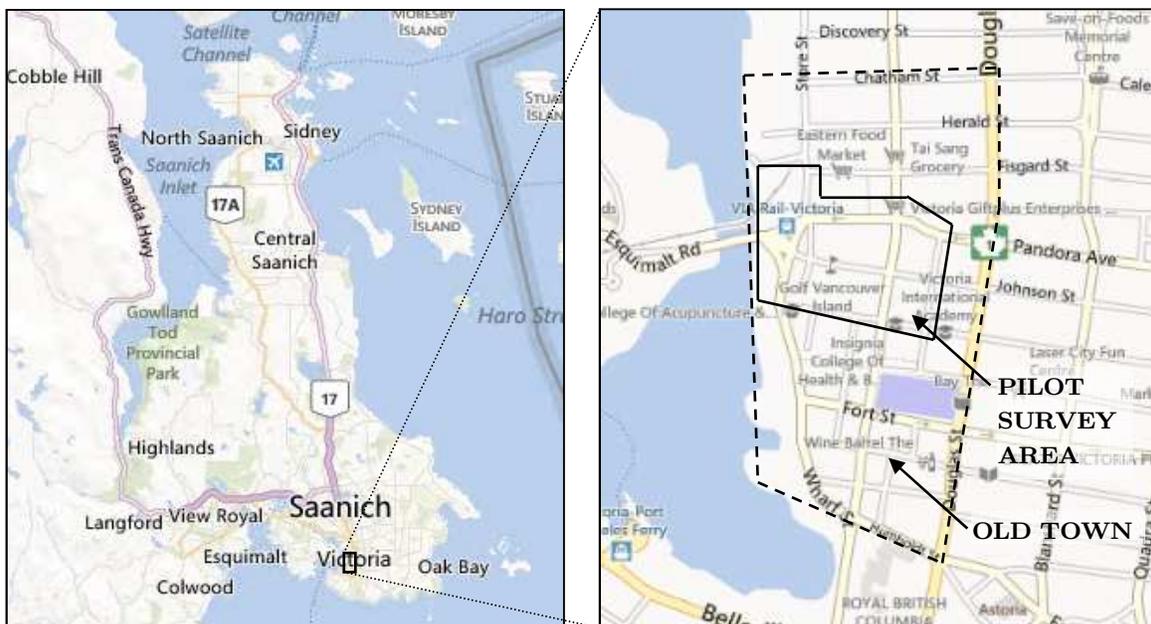


Figure 7.5 – Pilot Survey Area, Victoria, and “Old Town”

After: Paxton, Elwood, Barber, and Umland, 2013

7.2.2.1 Development of Data Collection Sheets

The first step in the process of developing the screening methodology was to create a data collection form that met the needs of our study. Several existing forms were reviewed (eg. Figures 6.1 and 6.2). Ultimately, an entirely new form was created, consisting of four sheets in total. As it was assumed that varying levels of access would be permitted (i.e. some building owners would welcome screeners into buildings while others would not), the forms were organized by level of access and a quick reference sheet was also provided. Figure 7.6 shows the “Exterior Access” sheet; the complete set is contained in Appendix F.

- 1) Exterior Access
- 2) Interior Access
- 3) Roof Access
- 4) Reference Sheet

The “Exterior Access” sheet is the most essential and can be completed by means of a typical ‘sidewalk survey’ type of review. Much of the information can also be gathered before going into the field by using available internet maps from Google, Bing, etc. The following types of information are recorded:

- Basic Building Information (eg. address, number of stories, floor area)
- Sketches/Photos
- Exterior Observations (eg. presence of adjacent buildings, presence of a veneer)
- Occupancy
- Exterior Areas (eg. sidewalks, seating areas) that could be subject to fall hazards
- Masonry Condition (scratch tests of mortar and bricks)
- Wall Height/Thickness
- Extent of Seismic Retrofitting (eg. braces, anchors)
- Fall Hazards (eg. presence, location, approximate height, areas below)
- Screening Score (using the methodology from Section 6.3)

The “Interior Access” sheet is supplementary in nature and provides for a more detailed account of items including: floor-by-floor occupancies, interior gravity framing, retrofit extents, and floor-by-floor wall height/thickness.

The “Roof Access” sheet is also supplementary in nature and provides for detailed recording of rooftop fall hazards such as parapets, cornices and chimneys. Space for sketches and measurement recordings are provided for all three items. Space to record the condition of the roofing is also provided.

Building Field Data Collection Form
Page 1 of 4

UBC Study:
URM Buildings Survey in Victoria, BC

Reviewers: _____ / _____
Date Completed: _____

PART 1 - EXTERIOR ACCESS

SKETCH OF OVERALL BUILDING		BASIC INFORMATION			
Plan View Photo/Sketch (show/label the adjacent streets)		Historic Address: _____ Bldg Name: _____ Year Built: _____ Postal Code: _____ Latitude: _____ Longitude: _____ Bldg Owner: _____ Site Class: _____ S ₁ (0,2): _____ Height [ft]: _____ Storeys Above Gr: _____ NZ Bldg Type: _____ Est. Flr Area [ft ²]: _____ Other Address(es): _____			
		GENERAL EXTERIOR			
Elevation View(s) Photo/Sketch (note which street viewed from)		Open front? (Y/N-show on elev sketch, eg. mostly glass): _____ LFRS @ open front? (steel braces, moment frame): _____ Adacent Buildings? (Y/N-dimension elev. sketch): _____ Shares wall w/ adj bldg? (Y/N-show on elev. sketch): _____ Adj bldg # storeys dif.? (Y/N-show on elev. sketch): _____ Unmatching Floors? (Y/N-show on elev. sketch): _____ End building? (Y/N-show on plan sketch): _____ Header courses every _____ courses Veneer? (Y/N, eg. weep holes, no header courses): _____			
		OCCUPANCY (circle all that apply)			
		High Density	Medium Density	Low Density	
		Assembly	Offices	Residential	Storage
		Mercantile	Institutional	Hotel	
		Personal Service	Manufacturing	Industrial	
		Restaurant/Bar			
EXTERIOR AREAS					
Lower Roofs of Adjacent Bldgs: _____ Sidewalks, Alleys? (Y/N-show on plan sketch): _____ Seating Area? (Y/N-show on plan sketch, eg. patio): _____					
MASONRY CONDITION					
Scratch Test Trial (show locations on sketch)					
		Trial 1	Trial 2	Trial 3	
Brick					
Mortar					
FEMA 154 SCORING					
Basic Score (URM)	1.8	FALL HAZARDS (use roof access instead if possible)			
Vertical Irreg.	-1.0	Type	Parapets	Cornices (Heavy)	Cornices (Light)
Plan Irregularity	-0.5	Present (Y/N)			Chimneys
Soil Type (Circle one)	A/B C D E/F	Elev. (N,S,E,W) Height (in) Braced? Areas Below (1,2,3)			
Bldg Vuln. (Circle one)	High Mod. Low	-0.1 - +0.1	Other fall hazards (describe):		
Ped. Density (Circle one)	High Mod. Low	-0.3 - +0.3	Notes:		
Occupant Density (Circle one)	High/Mod. Mod. Mod./Low Low	-0.3 -0.15 - +0.15 +0.3	- Cornices: wood/tin = light; terracotta/brick = heavy - Show fall hazards on sketches - For "Braced?": indicate Some/All/DNK - For "Areas Below": 1=sidewalk/alley, 2=adj. bldg, 3=low traffic		
Retrofit Extents (Circle one)	None Part'l Para. All Para. Tens. Ties Full	- +0.2 +0.6 +0.7 +1.0	FALL HAZARDS COMMENTS		
FINAL SCORE, S =		Note items such as deterioration, damage, mitigation work in progress			
EXT. WALL HEIGHT/THICKNESS (use interior instead if possible)					
		1st Storey	Typ. Storey	Top Storey	
Height (ft)					
Width (in)					
SIGNS OF SEISMIC RETROFIT (use interior instead if possible)					
Anchor plates visible @ floor levels? (Y/N): _____ Steel braces visible in windows? (Y/N): _____					
GENERAL COMMENTS					
Note items such as: additions (eg. wood storey); structural deterioration not noted above					

* = Estimated, subjective, or unreliable data
N/A = Not Applicable

DNK = Do Not Know

Dark Grey Shaded Heading = Minimum Info
Light Grey Shaded heading = Secondary Info

Figure 7.6 – “Exterior Access” Data Collection Sheet

Finally, the “Reference Sheet” contains information on potential structural deficiencies, masonry scratch testing, and example photos and figures for various items.

7.2.2.2 Data Collection Process

With the data sheets developed, the next step was to perform the survey of the buildings. The data was collected in two phases:

- Phase 1 included 48 buildings and was conducted in January-February of 2013
- Phase 2 included 33 buildings and was conducted in July-August of 2013

For Phase 1, the industry sponsor and the author partnered with two teams of undergraduate students at the University of British Columbia: one team of civil engineering students and one team of geography students. The student teams completed the surveys with co-ordination from the industry sponsor and training/supervision provided by the author. The results were then mapped by the geography students on a building-by-building basis. In Phase 2, the author performed the surveys; the maps were not updated to include these results.

In collecting the data, there were several important phases, as follows:

- 1) Pre-field data collection/review: sources such as online maps, existing soils maps, heritage registers, and past surveys were reviewed and pertinent information was recorded
- 2) Owners were contacted to request interior/roof access
- 3) Buildings were reviewed in the field and the results recorded on the data collection forms for each individual building; for student teams, typically 2 persons were assigned to each team
- 4) The information was input to a master database and reviewed for accuracy and consistency (eg. in scoring)

7.2.2.3 Results

There are several potential uses for the results of such a survey. However, the purpose of completing this pilot survey was primarily to develop and test the screening methodology of Chapter 6. The purpose was not to identify vulnerable buildings for risk mitigation and thus the results are not fit for that use. The results presented are limited and are considered preliminary. The intent is that the City of Victoria could use the procedures developed as input into a City-commissioned survey of its entire URM building stock.

The first interesting result is in classifying the building stock according to the New Zealand Typology, as developed by Russell and Ingham (2010). Figure 7.7 compares the results for samples from Christchurch (New Zealand), San Francisco (USA) and Victoria.

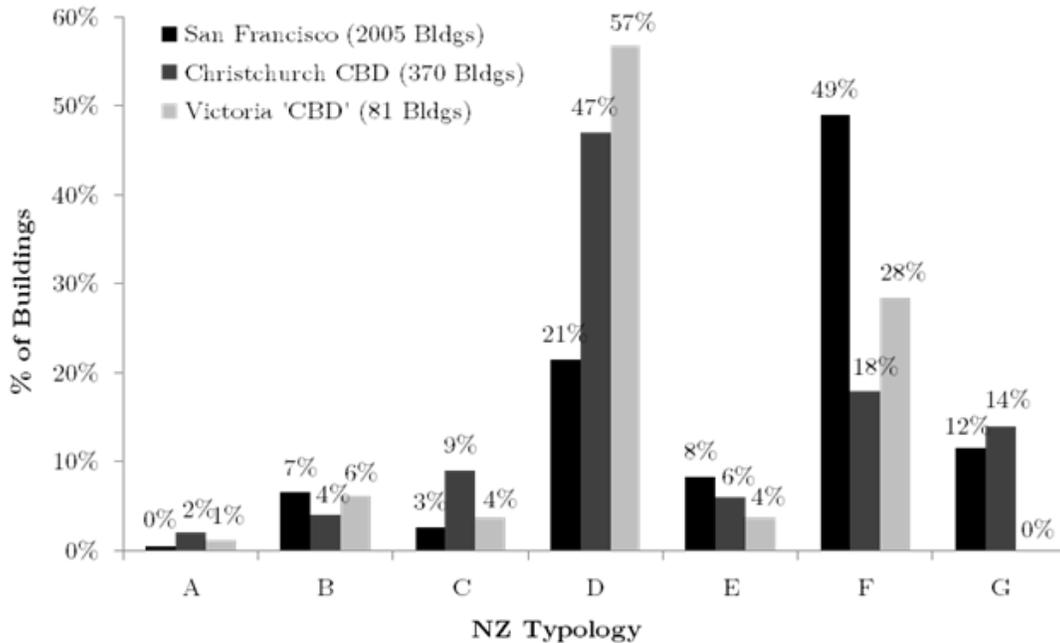


Figure 7.7 – Comparison of NZ Typology

Based on these results, Victoria’s building stock appears more similar to that of Christchurch than to that of San Francisco. However, it should be noted that the 81 buildings is only a sample of the URM buildings in the downtown core. The lack of any Type G buildings whatsoever is particularly conspicuous and is essentially due to the location of the pilot survey. To the north of the pilot survey area, there is a more industrial area that includes some buildings that would fall into this category.

Another result of interest is the observed retrofit rate. Five different levels were distinguished in the survey and are shown in Figure 7.8. The 21% “full” retrofit (i.e. comprehensive in-plane/out-of-plane) rate is a fair bit higher than the prevailing city-wide rate is thought to be. This is because the pilot area includes areas where seismic retrofit incentive programs have seen much use. Another observation is the distinct lack of partial retrofits – this was the impetus for the development of the “Parapet Incentive Program” by the industry sponsor. Of course, it should be noted that there could be additional retrofits that went undetected, as interior access was not provided for the majority of the buildings.

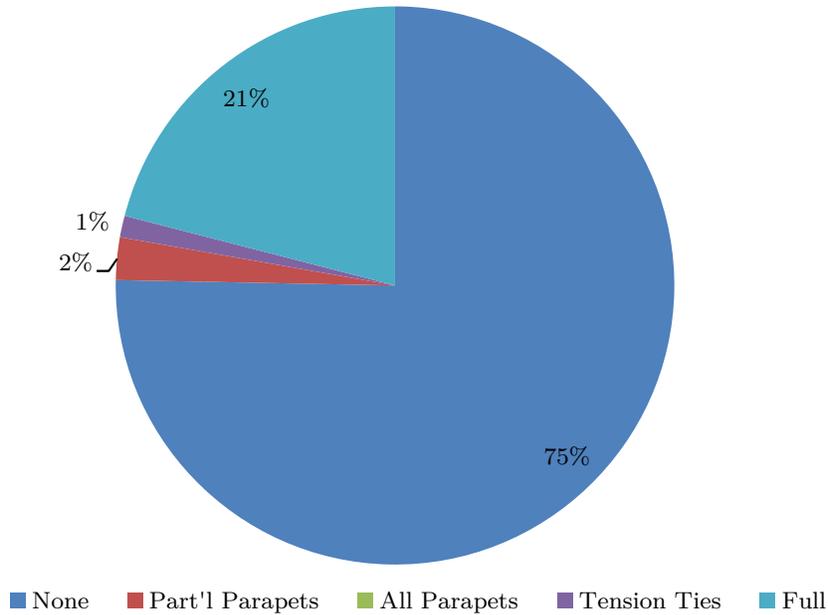


Figure 7.8 – Retrofit Rates in Victoria (81 Buildings)

Finally, it is most interesting to compare the results for the FEMA 154 scoring. Figure 7.9 shows the resulting distributions for the original scoring method and the modified method. As can be seen, the original method resulted in many identical scores, which made it ineffective at prioritizing *between* URM buildings. Recall that the original method was developed for the purpose of screening an overall population of buildings.

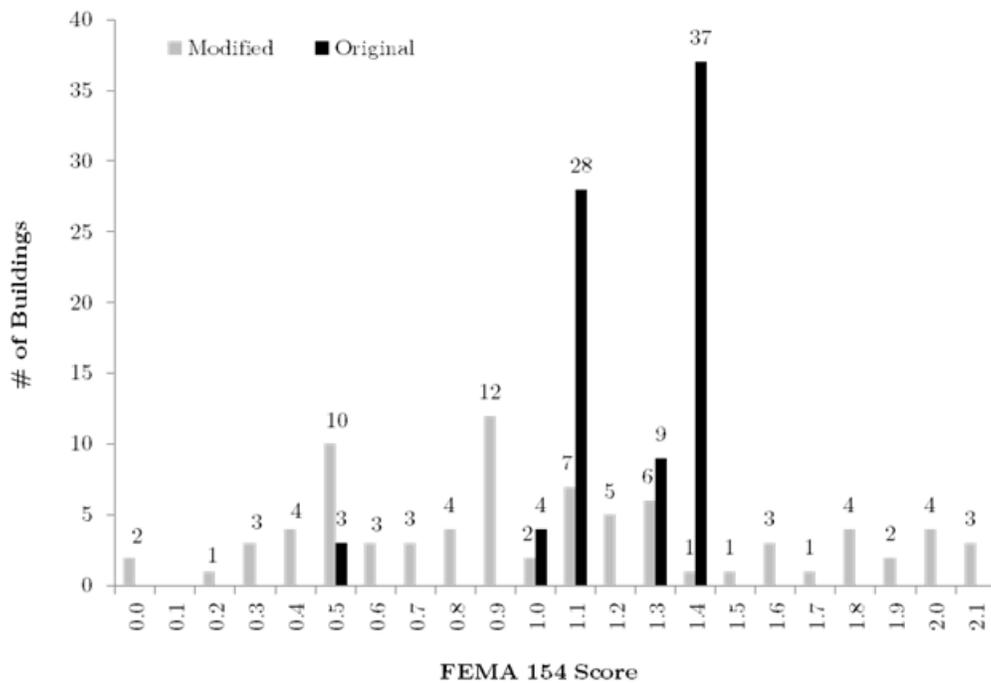


Figure 7.9 – FEMA 154 Scores, Original and Modified (81 Bldgs)

7.3 Conclusions

7.3.1 General Conclusions

It was shown herein that the original FEMA 154 scoring provided little resolution (soils were essentially the only distinguishing feature). The modified scoring procedure is better at distinguishing between URM buildings. For an overall survey of buildings, the original scoring procedure is recommended, as it is sufficient for this purpose. However, the modified procedure is recommended for a URM-only survey, or in addition to the original scoring method in an overall survey.

7.3.2 Conclusions for Victoria

Evidently, there has been interest in the past on the part of some City staff in identifying URM seismic risk by completing an inventory of URM buildings. However, interest appears to have waned and the inventory has become outdated. The City of Victoria should commission a survey of its URM buildings; the methodology developed and applied herein is one possible example.

Chapter 8

Summary and Conclusions

A great variety of issues have been addressed throughout this thesis. This chapter highlights the most significant achievements and the conclusions that were reached.

8.1 URM Buildings and Risk Mitigation Efforts in Victoria, BC

In Chapter 2, it was shown that URM buildings¹¹ and the related seismic rehabilitation techniques common to Victoria are similar in form to others throughout the west coast of North America. URM bearing wall buildings and seismic rehabilitation techniques in New Zealand were also found to be somewhat similar in form.

In Chapter 3, URM seismic risk mitigation efforts throughout the west coast of the U.S.A. and New Zealand were reviewed and compared those of Victoria. It was found that most other regions had adopted risk reduction policies and developed programs to address URM buildings: the State of California passed the “URM Law” in 1986 and many cities ultimately adopted mandatory strengthening/abatement ordinances. The result is that in cities such as Los Angeles and San Francisco, nearly 90% of URM buildings have been mitigated, through either strengthening or demolition (note: this figure includes other types of URM, although bearing wall buildings are by far the most common). Seattle, Washington is also in the midst of developing a mandatory strengthening ordinance and Portland, Oregon is also considering developing one.

Some success has been achieved in Victoria through the current heritage incentive funding programs, but efforts overall were found to be lacking. It appears that only about 10-15% of URM buildings in Victoria have been comprehensively seismically upgraded as a result of re-development and the associated heritage incentive programs. Moreover, there are currently no incentives or city requirements specifically targeting partial retrofitting of buildings¹². This is of vital importance as many buildings are not

¹¹ Recall that URM infill and confined masonry buildings are not common in Victoria and so the term “URM” in this thesis refers strictly to bearing wall buildings. Other types of URM buildings (eg. concrete/steel frame with URM infill) are specifically identified as such, as needed.

¹² In December 2014, just days before the final submission of this thesis, the Victoria Civic Heritage Trust approved the aforementioned “Parapet Incentive Program” (see Section 3.5.2.3), which will help to fulfill the need for partial retrofitting measures in Victoria.

viable candidates for redevelopment, either due to their form or owner finances. Finally, there is currently no comprehensive (and up-to-date) inventory of URM buildings, which is a necessary first step for all communities looking to mitigate URM seismic risk.

8.2 Quantifying Building Vulnerability Through Observed Damage

Throughout the review of seismic risk mitigation efforts abroad, it became apparent that most risk mitigation efforts were undertaken in response to past earthquake losses. To provide a more rational basis for promoting earthquake strengthening, it was decided to quantify building performance through observed damage statistics. This empirical approach was selected because current analytical methodologies such as HAZUS fail to recognize important differences in dynamic behavior between URM buildings and most other modern buildings, as discussed in Chapter 2. The use of observed damage was feasible because damage data had been collected by others for a number of significant earthquakes, including the 1987 Whittier Narrows, 1989 Loma Prieta, 1994 Northridge, and 2010/2011 Canterbury (New Zealand) earthquakes, as presented in Section 4.7. Data from the Loma Prieta and Northridge earthquakes was provided to the author by Mr. Bret Lizundia of Rutherford and Chekene Consulting Engineers, and a database from the 2010/2011 Canterbury earthquakes was provided to the author by Dr. Jason Ingham of the University of Auckland, New Zealand.

As discussed in Section 4.8, motion-structural damage relationships were developed for four strengthening statuses for the various databases:

- Unretrofitted
- Parapets-braced
- Partially-retrofitted
- Fully retrofitted

As expected, the results indicated that limited retrofits (eg. parapet bracing, tension ties) achieve nearly the same results as full retrofits for low intensity shaking; however, the improvements are marginalized at higher intensity shaking. The results from the various data sets were compared and it was found that the Canterbury buildings appeared to be more vulnerable than the California buildings (for various reasons as discussed in Section 4.8.5). However, the variation decreased with increased strengthening.

As the ultimate goal of the study was to develop cost-benefit analyses for Victoria, motion-structural damage relationships were defined for Victoria, based on weighted averages of the California (i.e. Loma Prieta and Northridge) and the New Zealand (i.e.

2010/2011 Canterbury) data. As discussed in Section 4.9, lower bound, best, and upper bound estimates were defined as:

- Lower bound: 50%/50% weights on the California and Canterbury, respectively
- Best Estimate: 33%/67% weights on California and Canterbury, respectively
- Upper Bound: 100% weight on the Canterbury data

The rationale for the lower weight on the California data was the fact that it did not include higher intensity shaking (as did the Canterbury data) and placing increased weight on an extrapolation seemed imprudent.

8.2.1 General Conclusions

- 1) It was found that there can be significant differences in motion-damage relationships for “similar” types of buildings; this is thought to be primarily due to differences in construction and in seismic demands (beyond that which is captured by the chosen intensity measure)
- 2) It was found that limited strengthening measures such as parapet bracing and tension ties provide similar degrees of damage reduction to that of full retrofits under low intensity shaking, but that the improvements are marginalized with increased shaking
- 3) Significant differences were present between the observed motion-damage relationships and those from previously published sources (ATC 1985, EERI 1994). Published sources regularly appeared to overestimate damage; this same conclusion was reached by Rutherford and Chekene (1997)
- 4) The new results are thought to be an improvement over previously available results, particularly because the new results address buildings with parapet-only retrofits and because the data includes samples at higher ground shaking intensities (i.e. from the Canterbury earthquake sequence)

8.2.2 Conclusions for Victoria, BC

- 1) It appears that the vulnerability of Victoria’s URM buildings falls between the vulnerability of those in Canterbury and California
- 2) Parapet bracing and partial retrofits can offer significant overall damage reduction (as well as improved life-safety) for low-intensity shaking (eg. $S_a(1) \leq 0.15g$)
- 3) For more intense shaking (eg. $S_a(1) \geq 0.4-0.6g$), such as is plausible for Victoria, the benefits of parapet partial strengthening are marginalized

8.3 Cost-Benefit Analysis for URM Seismic Rehabilitation in Victoria

The economic viability of URM seismic retrofitting was investigated by means of cost-benefit analysis. Specifically, the costs/benefits for seismic upgrading of a “typical” commercial occupancy building in downtown Victoria were quantified. The assessment was probabilistic in nature, with results in terms of annual expected costs.

The benefits of seismic strengthening lie in the reduced expected value of earthquake losses. The losses (and thus benefits) considered included:

- Building damage (owner benefit)
- Occupant casualties (public benefit)
- Pedestrian casualties (public benefit)
- Tenant relocation expenses (owner benefit)
- Lost rental income due to downtime (owner benefit)

The cost, of course, is the design and construction cost of the proposed seismic strengthening. To complete the analysis, the expected benefits (i.e. reduced expected losses) over a time horizon of 50 years (2014-2064) were calculated and converted to their present value. Discount rates of 5% and 3% were applied to the owner and public benefits, respectively. Sensitivity analyses were conducted for several parameters, including the cost of retrofitting, discount rates, streetfront (i.e. pedestrian) exposure.

8.3.1 General Conclusions

With regards to the probabilistic cost/benefit analysis, the results (based on both public and owner benefits) indicated the following:

- 1) Parapet bracing appears to be a good to excellent investment for the vast majority of buildings, having benefit-cost ratios (BCRs) of 0.79 (Site Class B) to 4.03 (Site Class E)
- 2) Partial retrofits appear to be good investments for buildings on soft soils, with BCRs of 0.87 for Site Class D and 1.69 for Site Class E
- 3) Full retrofits appeared not to be good investments regardless of soil type, with a maximum BCR of 0.68 for Site Class E; this result supports the common practice of only requiring “full” retrofits when a building is undergoes a substantial renovation (for other reasons) or a change of occupancy
- 4) When considering only the owner benefits, retrofitting was generally not economically favorable; this points to the need for incentives and financial assistance programs

Based on the earlier discussions (see Chapter 5), it is clear that expected values of costs/benefits are not the only possible considerations in decisions on earthquake strengthening. Recall also that some effects (such as risk-averseness) and some benefits (such as preservation of heritage) were not considered in the cost-benefit analysis. Overall, it is felt that a BCR should serve as a first pass indicator, in that any result above 1.0 should be conclusive, while BCR's above about 0.8 to 1.0 are possible candidates, depending on the level of risk averseness and the value placed on heritage preservation.

8.3.2 Conclusions for Victoria, BC

With regards to URM seismic risk mitigation in Victoria, the following conclusions were reached:

- 1) An incentive program for parapet bracing or partial retrofitting appears to be a good use of public money
- 2) Parapet bracing achieves a significant portion (approximately 70%) of the expected casualty reduction of a full retrofit. This is because it is highly effective at reducing casualties in the low-intensity (high-probability of occurrence) region of the hazard curve. Of course, the benefits eventually disappear at higher levels of shaking, but the probability of occurrence is low for such events
- 3) There is good evidence for cost sharing among building owners and the public, since about 40-60% of the benefits lie in reduced casualties (i.e. improved public safety)

A final item worth pointing out is that *earthquake insurance is not a substitute for strengthening*, or vice-versa. As discussed in Section 5.9.3.2, only about 25% of the expected losses for an unretrofitted are eliminated by an insurance policy with a 10% deductible.

8.4 Methodologies for Assessing and Prioritizing URM Seismic Risk

Several existing methodologies were reviewed, including ad-hoc methods developed by individual cities and more general, scientific methods including FEMA 154 and a related Canadian methodology (NRC 1993).

Ultimately, it was concluded that neither of the two general methodologies were particularly well-suited for a URM-only survey. Various modifications were developed for URM buildings. New FEMA 154-compatible score modifiers were proposed,

addressing the effects of strengthening, pedestrian and occupant density, and URM building typology (as discussed in Section 6.3).

8.5 Inventory and Screening of Victoria’s URM Buildings

An existing, but dated (circa 1989) survey of buildings in Old Town was reviewed. The survey included 329 buildings, 260 (79%) of which were deemed to be of URM construction. A separate survey was compiled by the author, with the assistance of several other parties including teams of University of British Columbia (UBC) undergraduate students, the City of Victoria GIS department, and the industry sponsor, as discussed in Section 7.2.2. The FEMA 154 screening methodology (with and without the author’s proposed modifications) was applied in rating the buildings. As intended, the refined FEMA 154 methodology provided much more “resolution” in the results. However, it should be recalled that in developing the modifications, no effort was made to ensure compatibility with the overall FEMA 154 framework, so the modifications are only currently considered appropriate for URM-only surveys.

8.6 Objectives Achieved

Section 1.4 listed five high-level objectives that were defined in co-operation with the industry sponsor. These objectives are revisited here:

- 1) To gain an improved understanding of the seismic risk due to unreinforced masonry buildings in “Old Town” (an area of Victoria’s downtown core)
- 2) To develop material for educating stakeholders about the risks
- 3) To develop a rational and scientific basis for future work by VCHT in encouraging seismic upgrading through existing and new incentive programs
- 4) To develop a rational and scientific basis for future work by city officials in developing a URM seismic risk mitigation program
- 5) To develop methodologies that can be extended to other communities

It is felt that all five of these objectives have been fulfilled in this study. An improved understanding of the seismic risk has been achieved through the building surveys in Victoria, as well as producing the motion-damage relationships and applying them in estimating losses. Material for educating stakeholders is contained in essentially every chapter. A rational and scientific basis for future works in encouraging upgrading has been established through the cost-benefit analysis in Chapter 5. A rational and scientific basis for future work by city officials in developing a seismic risk mitigation program was

established in Chapters 3 and 6. Finally, the cost-benefit of Chapter 5 and the screening methodology of Chapters 6 and 7 can certainly be applied to other communities.

8.7 Contributions

This thesis has made contributions to the body of knowledge in the fields of URM seismic performance, seismic risk analysis, and seismic risk mitigation strategies, including the following:

- Damage statistics were examined for braced parapet buildings – this is something that has previously received little attention in the literature, presumably because parapet retrofit ordinances were already in place in many communities
- Damage statistics for the Canterbury earthquakes were analyzed and compared to observed damage statistics for several earthquakes – this study is thought to be one of the more comprehensive attempts in the literature for the subject buildings
- Structural motion-damage relationships compatible with HAZUS were generated. While there was much uncertainty in developing these, they are thought to be a significant improvement over previously available results (i.e. standard relationships in ATC-13 or HAZUS)
- Cost-benefit analysis for URM seismic rehabilitation in Victoria, BC was performed – this study is certainly the most comprehensive Canadian treatment of the issue
- The cost-benefit methodology that was developed could be applied to other areas facing similar risk. The following issues are considered
 - Similar to above, this study is one of few to look at parapet bracing
 - This study made a preliminary attempt to account for “scale effects” on downtime (such as was observed in the Christchurch CBD), which is not captured in HAZUS or other loss estimation methodologies
 - This study attempted to account for the effects of structural collapse on losses to non-structural components and building contents
- The effectiveness of FEMA 154 for URM-only surveys was assessed and modifications to the methodology were proposed to help distinguish amongst URM buildings. This is valuable for many communities looking to assess and prioritize their URM seismic risk, as FEMA 154 is commonly used

8.8 Future Research Opportunities

- Damageability effects of various building characteristics (eg. number of stories) as discussed in Section 4.11.3
- Improved estimates of downtime. Data collected in response to earthquakes has typically focused much more on building damage than downtime. However, a study by Comerio (2006) provides a strong example. A framework has been proposed in this study and future research calibrating the parameters would be valuable
- Improved fragility of non-structural components would be highly valuable. Default data from HAZUS was used in this thesis (with some adjustments), but it was noted that the non-structural damage patterns could conceivably be quite different in URM buildings with flexible diaphragms (eg. perhaps one would expect more damage to floor and ceiling finishes and less damage to partitions and/or windows)
- Various refinements specific to Victoria could be made by using locally-collected data such as pedestrian density and building data
- Scenario loss estimates were not completed herein, primarily due to the lack of an up to date and complete database. Scenario loss estimates are more tangible and would likely be useful to decision-makers
- Although the FEMA 154 methodology is quite rational, it is based only on one particular seismic hazard level, towards the lower probability end of the hazard curve. A method based on the full extent of the hazard curve may be more scientifically justified and may be more useful for owners looking to prioritize risk based on expected cost

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Appendix A

Costs for URM Seismic Rehabilitation in Victoria, BC

A.1 Summary

This appendix provides unit costs for common URM seismic rehabilitation activities, in 2014 Canadian dollars. The purpose for the work was two-fold: firstly, the Victoria Civic Heritage Trust is the process of developing a new incentive grant program for partial upgrading as discussed in Chapter 3 and an estimate of the potential costs for such work is needed in order to select an appropriate dollar value for grants. Secondly, costs for seismic upgrading measures were of course necessary in order to undertake the cost-benefit analysis, as presented in Chapter 5. The measures vary in scope from targeted measures, such as parapet bracing and veneer anchoring, to comprehensive seismic upgrading, in accordance with current Canadian design standards.

A variety of sources were considered in developing the estimated costs, as enumerated below.

- 1) Costs for projects completed in Victoria, BC
- 2) Itemized estimates for specific upgrade details, based on published component costs from the RSMeans Building Construction Cost Data 2012 catalog
- 3) Published unit costs, primarily based on surveys of projects completed in California, U.S.A.

As will be discussed herein, there is substantial uncertainty in deriving unit costs and each source has merits and drawbacks. The sources are listed in terms of the author's recommended preference for use. For example, the Detailed Estimate value for "Parapet Bracing (Thru-Bolted)" should be given preference over the Published Survey value; the latter is simply provided to give the reader a sense of

the potential range of costs, since there can be substantial differences in the scope of work even for a specific upgrading activity.

The following values were used in the cost benefit analysis, drawing from the information presented within this appendix.

Table A.1 Summary of Unit Costs for Rehabilitation Measures

Upgrade Description	Unit Costs Derived From		
	Victoria Projects (Pref. #1)	Detailed Estimates (Pref. #2)	Published Surveys (Pref. #3)
<i>C.2 - Targeted Upgrades</i>			
Parapet Bracing, Tall Parapet (Thru-Bolted)	--	\$274/LF	\$161/LF
Parapet Bracing, Short Parapet (Thru-Bolted)	--	\$142/LF	--
Parapet Bracing, Tall Parapet (Epoxy Anchors)	--	\$429/LF	--
Parapet Bracing, Short Parapet (Epoxy Anchors)	--	\$281/LF	--
Veneer Anchors (Epoxy Dowels)	--	\$14/SF	--
Veneer Anchors (Expansion Anchors)	--	\$10/SF	--
Wall Tension Anchors (Thru-Bolted)	--	\$86/LF	--
Wall Tension Anchors (Epoxy)	--	\$120/LF	\$202/LF
<i>C.3 - Intermediate Upgrades</i>			
Bolts-Plus (Wall Anchors + OoP Bracing)	--	--	\$22/SF
<i>C.4 - Comprehensive Upgrades</i>			
Upgrade To High Seismic Zone Retrofit Standards	\$40/SF	--	\$35/SF

¹ Costs shown are for construction only; taxes and soft costs not included; refer to specific sections for information on potential total costs

² For epoxy anchors, the URM wall must be 3-wythes wide; 2-wythe parapets are not uncommon; 2-wythe walls are not common in Victoria, but have been reported in other regions

A.2 Targeted Upgrades

Fall hazard mitigation & wall anchorage are commonly the first targeted upgrade in a URM seismic risk mitigation scheme because they are considered among the most vulnerable components and least expensive to upgrade (Rutherford & Chekene, 1997). As discussed in Chapter 3, several jurisdictions in the Pacific Northwest long ago adopted parapet-related seismic risk mitigation ordinances.

A.2.1 Unit Costs from Published Sources

A great deal of work on URM seismic rehabilitation, including quantification of costs, was undertaken from the late 1980's to mid-1990's in the United States, primarily based on California data. The impetus for this was an ongoing movement towards mandatory URM seismic rehabilitation, including California's 1986 URM

Law (Paxton, Elwood, Barber, & Umland, 2013), as discussed in Chapter 3. Although most of the efforts focused on higher levels of seismic upgrading (since most of the affected communities had long ago implemented parapet ordinances), two publications were found to explicitly provide historical unit costs for parapet bracing (Wong, 1987; FEMA, 1988).

“Seismic Strengthening of Unreinforced Masonry Buildings – A Design Guide for Architects and Engineers” (Wong, 1987) was published by the US National Science Foundation in 1987 and provides unit costs for various common URM retrofit details including parapet bracing. Although quite dated, URM retrofit standards (including force levels) have changed relatively little since this time and thus the costs should be representative, provided one properly accounts for the time effects (inflation, changes in labour pricing, etc.). The detail for parapet bracing is reproduced as Figure A.1. The cost for this work is listed at \$55/LF (in 1987 USD). Adjusting this cost from 1987 USD to 1987 CAD and converting from 1987 dollars to 2014 using RSMMeans’ historical cost indices (Reed Construction Data, 2012) yields a cost of **\$161/LF** as shown below. Note that this does not include sheathing replacement, re-roofing, or repointing; however, this additional work is accounted for in the detailed estimates (see Section A.2.2).

$$\text{Wong 1987 cost converted to 2014 CAD: } \$55/\text{LF} * \frac{1 \text{ CAD (1987)}}{.75 \text{ US (1987)}} * \frac{100 (2014)}{45.5 (1987)} = \mathbf{\$161/\text{LF}}$$

The second publication is FEMA 156 (1988). This document provides square-foot costs for various rehabilitation activities, based on a survey of just 199 buildings, of which 137 were located in the City of Los Angeles. The costs presumably represent US Dollars, between 1981 and 1988, since Los Angeles brought its mandatory URM retrofit ordinance into effect in 1981.

The document lists an average cost of \$0.58/SF for parapet bracing; this is not a particularly useful number since a square-foot cost for parapet bracing would be highly dependent on the number of storeys. A large fraction of the buildings surveyed were 1-storey commercial buildings, so perhaps one could assume use the figure based on the building footprint area. Another issue is that the study showed

that the cost of URM rehabilitation work varied greatly from city to city: the average cost of overall upgrading in Los Angeles was just \$6.40/SF, compared to \$14.60/SF in Long Beach and \$34.00/SF in cities that did not have a URM retrofit ordinance. This variation is postulated to be due primarily to three sources: differences among building forms, design practices, and the familiarity of the contractors with the work. In spite of these uncertainties, one may rationalize appropriateness of the unit cost since the buildings were primarily 1-storey and parapet bracing would be largely unaffected by occupancy, location, or contractor experience, since the design standards and resulting scopes of work do not vary greatly for this particular item. Adjusting this cost from USD to CAD and from 1987 to 2014 yields a cost of **\$1.70/SF** (where SF is the footprint area, not the gross floor area). Based on the aforementioned issues, however, this estimate is not thought to be particularly reliable. Preference should be given to other cost estimates.

$$\text{FEMA cost converted to 2014 CAD: } \frac{\$0.58}{\text{SF}} * \frac{1 \text{ CAD (1987)}}{.75 \text{ US (1987)}} * \frac{100 (2014)}{45.5 (1987)} = \mathbf{\$1.70/SF}$$

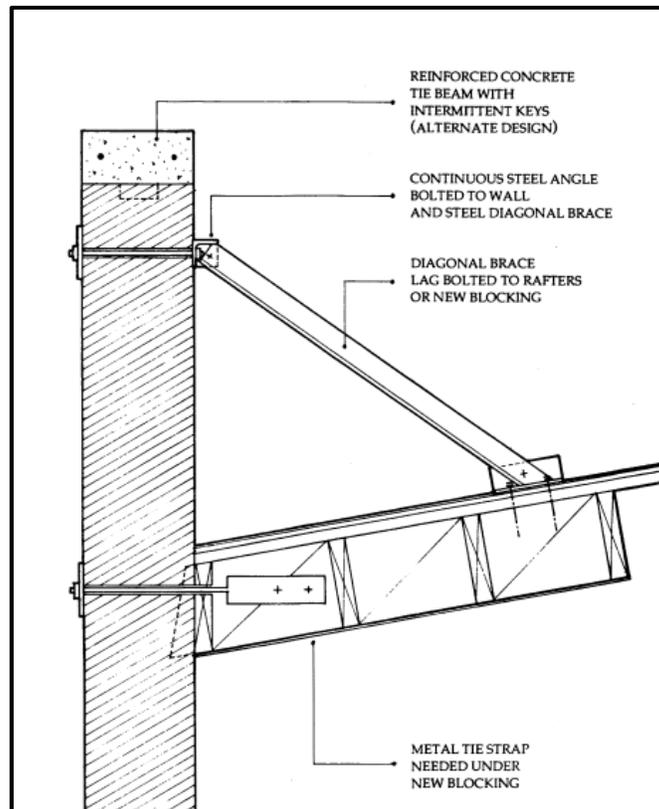


Figure A.1 - Parapet Bracing Detail (From Wong, 1987)

A.2.2 Unit Costs from Detail-Specific Estimates

Although deriving costs from completed projects has value in that it implicitly reflects the market conditions and complete scope of work for the projects in question, this is also a potential pitfall: the market conditions and scope of work for the historical projects may not be representative of the desired conditions – in this case, present-day Victoria. One method to overcome this is to complete a component-by-component estimate, for an explicitly defined scope of work (i.e. a design detail), similar to what would be performed by a contractor submitting a bid to perform the work.

By breaking down the scope of work into various components (eg. structural steel braces, wood sheathing) one can account for the varying conditions: for example, the material cost of structural steel in Victoria may be above average, since it is a relatively small community located on an island, while the labour cost to install the steel may be below average because the cost of living in the smaller community is lower than for a larger community. Of course, a great deal of data and a formal procedure is required to perform such an estimate. The *RSMMeans Building Construction Cost Guide* (Reed Construction Data, 2012) is such a publication, including detailed cost data based on U.S. national averages, which are then converted to the specific conditions through various adjustment factors.

Detail-specific estimates were completed in this manner for two parapet bracing designs, and two veneer anchorage designs, and two wall tension anchor designs. Table A.2 summarizes the unit costs for the various details; the “Total Cost” column includes 15% for soft costs and 5% for taxes. The individual estimates are presented at the end of this appendix.

Table A.2 Unit Costs for Targeted Seismic Upgrades in Victoria

Det. #	Description	Constr. Cost	Total Cost
<i>Parapet Bracing Details</i>			
1a	Thru-Bolt Tension Anchors (Tall Parapet)	\$274/LF	\$331/LF
1b	Thru-Bolt Tension Anchors (Short Parapet)	\$142/LF	\$171/LF
2a	Epoxy Shear and Tension Anchor (Tall Parapet)	\$429/LF	\$518/LF
2b	Epoxy Shear and Tension Anchors (Short Parapet)	\$281/LF	\$339/LF
<i>Veneer Retrofit Details</i>			
1	Epoxyed Dowels in Mortar Joint From Exterior	\$14/SF	\$17/SF
2	Expansion Anchors in Mortar Joint From Exterior	\$10/SF	\$13/SF
<i>Wall Tension Anchor Details</i>			
1	Thru-Bolted Anchors, New Sheathing & Flooring Re and Re	\$86/LF	\$104/LF
2	Epoxy Anchors, New Sheathing & Flooring Re and Re	\$120/LF	\$145/LF

For the parapet upgrades, there is clearly a great deal of variation amongst the various details, illustrating the aforementioned pitfall in using historical data. Parapet detail 1a would be considered the most the representative of a typical parapet upgrade and is most consistent with the cost detail from (Wong, 1987). Wong’s cost did not include re-pointing or re-roofing, as this was covered under other items; it also did not include value-added taxes or soft costs. Removing these components from the estimate for parapet detail 1a yields a unit cost of **\$146.58/LF**. This compares quite well to the previous unit cost of **\$161/LF** provided by Wong.

Note that for all the above epoxy anchor-type upgrades, the wall being anchored must be at least 3 wythes wide. Two-wythe parapets are not unusual in Victoria, although three-wythe parapets are likely still more prevalent. Two-wythe walls (i.e. at any floor below the roof) are uncommon in Victoria; however, they have been reported in other regions such as California (Rutherford & Chekene, 1997) and New Zealand (Ingham & Griffith, 2011).

A.3 Intermediate Upgrades

An intermediate level of seismic upgrading often considered involves measures to tie the entire building (i.e. all floors and walls), so that it responds as a unit. It may

also include bracing walls for out-of-plane demands. The intent of such upgrading is primarily to reduce the vulnerability to out-of-plane collapse of the URM walls. Generally, this level stops short of strengthening elements such as walls and diaphragms for in-plane demands, because such measures generally require that the building be unoccupied.

A.3.1 The ‘Bolts-Plus’ Approach

Perhaps the most well-known intermediate upgrade approach is San Francisco’s Ordinance 225-92. The ordinance, which mandated retrofitting/abatement of approximately 2000 identified URM bearing wall buildings, included a relaxation for buildings not containing certain occupancies (assembly >300; educational; institutional; or high-hazard) and having regular configuration, sufficient crosswalls, and percentage of solid wall (Paxton, Elwood, Barber, & Umland, 2013). This “bolts-plus” relaxation called for just diaphragm to wall connections (for shear and tension) and out-of-plane bracing for walls exceeding specified height-to-thickness ratios.

A study by Rutherford and Chekene (1990), which served as input in the city’s development of Ordinance 225-92, estimated the cost of this type of work for 15 different building prototypes (eg. differing size and occupancy). Based on the author’s judgment, the most likely candidate for this type of work are two and three storey commercial buildings (Prototypes G and H in the original publication). They are good candidates for the following reasons:

- They are common throughout Victoria
- They are unlikely to be unoccupied for an extended period of time because of their prominent location in downtown core, which would be required for more comprehensive upgrades
- They are unlikely to undergo a change of use or occupancy which would trigger requirements for a more comprehensive upgrade
- They are located in an area with a relatively high concentration of pedestrians, which are affected by the life-safety hazards from fall debris such as parapets and walls

The difference between the two prototypes is essentially their size, which has a significant effect on their cost per square foot. Rutherford and Chekene's (1990) estimates have been adjusted to 2014 Canadian dollars (using the 1990 CAD exchange rate of 0.85 and an inflation index of 2.04), and are provided below

- **Prototype G** (Small area, 2-3 St., Commercial): **\$29.62/SF**
- **Prototype H** (Large area, 2-3 St., Commercial): **\$13.35/SF**

These figures provide reasonable bounds for the Bolts-plus type upgrading work. One could average the two figures, yielding a value of **\$21.49/SF**, although this assumes a 50/50 split between large and small area buildings, which may not be the case.

A.4 Comprehensive Seismic Upgrading

Comprehensive seismic upgrading refers to an upgrade program that provides a complete seismic force resisting system, in accordance with current Canadian standards for seismic rehabilitation for URM buildings. For a location with a high seismic hazard, such as Victoria, this would typically include the measures noted below. Refer to *Appendix B – Typical Rehabilitation Measures* for a more detailed discussion.

- Repointing deteriorated or deficient masonry
- Mitigating fall hazards (eg. parapets, chimneys)
- Providing wall-to-diaphragm tension anchors
- Providing wall-to-diaphragm shear anchors
- Out-of-plane bracing for URM walls which exceed certain height-thickness ratios (eg. strongbacks)
- Providing new elements to resist in-plane loads where URM walls are absent or insufficient (eg. steel braced frames, concrete overlays or shear walls)
- Strengthening floor/roof diaphragms to reduce deflections (eg. plywood overlay)

Note that some items would not be required for areas of lower seismicity under current Canadian and American design standards.

A.4.1 Square-foot costs from Victoria projects

As discussed in Chapter 3, the City of Victoria has implemented its Tax Incentive Program, which promotes adaptive re-use of its heritage buildings by offsetting the costs of the comprehensive seismic upgrading required under the BC Building Code due to the change occupancy (Paxton, Elwood, Barber, & Umland, 2013). Because the cost of the seismic upgrading work is a parameter in determining the value of the incentive for each building, the cost of the work for each building is well-documented.

Costs for 34 buildings were available in total; the costs represent construction cost of the work, and exclude value added taxes and soft costs such as design and permit fees. Square footage for each building was estimated based on the number of storeys and the building footprint. Table A.3 provides the data for each building and the summary statistics. As can be seen, the mean and median costs are quite similar, at \$39.76/sq.ft and \$37.26/sq.ft, respectively. Of course, there is significant variation in the costs; there are a number of factors that could reasonably contribute to this including the following:

- The form of the building (short and squat, tall and slender, isolated vs. row)
- The economy at the time of construction
- The design standard employed
- The importance of the building
- The local soils at the building site
- The experience and judgment of the designers
- The heritage/architectural sensitivity of the rehabilitation

Table A.3 - Costs for Comprehensive Seismic Upgrades in Victoria

Bldg #	Project Cost	Seismic Cost	%Cost Seismic	Seismic Cost/sq.ft	# Storeys	Total Sq. Ft.
1	\$1,262,883	\$343,668	21%			
2	\$412,406					
3	\$1,851,360	\$414,964	22%	\$14.82	4	28000
4	\$2,000,000	\$317,559	16%	\$15.18	3	20922
5	\$1,518,725	\$407,921	27%	\$45.32	3	9000
6	\$845,580	\$283,558	34%	\$44.17	3	6420
7	\$408,891	\$176,968	43%	\$38.81	3	4560
8	\$6,000,000	\$2,171,520	36%	\$128.64	2	16880
9	\$550,000	\$212,000	39%	\$18.76	2	11300
10	\$2,000,000	\$370,000	19%	\$63.57	2	5820
11	\$2,206,087	\$440,967	20%			
12	\$9,500,000	\$1,400,000	15%	\$21.03	4	66560
13	\$400,000	\$310,523	78%	\$69.01	3	4500
14	\$3,400,000	\$1,025,425	30%	\$44.97	3	22800
15	\$8,000,000	\$1,055,000	13%			
16	\$10,936,927	\$1,265,069	12%	\$39.53	3	32000
17	\$10,700,000	\$7,048,000	66%			0
18	\$4,465,000					0
19	\$2,955,928	\$752,698	25%	\$37.26	3	20200
20	\$952,280	\$149,528	16%	\$17.39	2	8600
21	\$1,700,160	\$318,780	19%			0
22	\$3,099,840	\$581,220	19%			0
23	\$4,515,000	\$412,162	9%	\$17.61	3	23400
24	\$1,210,000	\$143,837	12%	\$27.66	2	5200
25	\$4,325,703	\$1,871,991	43%	\$63.67	7	29400
26	\$3,348,000	\$400,000	12%	\$25.81	5	15500
27	\$910,697	\$365,357	40%			0
28	\$5,500,000	\$488,045	9%	\$22.18	3	22000
29	\$4,113,560	\$2,321,253	56%			0
Mean	\$3,416,863	\$927,704	28%	\$40	3.16	14122
Med.	\$2,581,007	\$413,563	21%	\$38	3.00	10150
CoV	0.89	1.47	0.64	0.69	0.38	1.07

A.4.2 Square-foot costs from Published Sources

Several of the aforementioned sources have quantified the cost of comprehensive upgrading. FEMA 156 (1988) contained survey data for various locations such as Los Angeles and Long Beach, California. Rutherford and Chekene (1990) provided estimates for comprehensive upgrades to two different design standards for San Francisco’s URM buildings and Rutherford and Chekene (1997) provided estimates for upgrading to American standards (similar to current Canadian URM retrofit standards) for moderate vs. high seismic zones. Table A.4 summarizes the figures from various sources.

Table A.4 – Published Costs for Comprehensive Seismic Upgrades

Category	Design Standard ⁴	Seismicity	Mandatory Retrofits	Unit Cost ²	Source ³
Los Angeles, California (Division 88)	ABK	High	Yes	\$18.75/SF	1
Long Beach, California	TBC	High	Yes	\$42.78/SF	1
Cities Without Ordinances	TBC	High	No	\$99.63/SF	1
Schools in California	TBC	High	Yes	\$127.79/SF	1
San Francisco (Prot. G), UCBC	ABK	High	Yes ¹	\$32.86/SF	2
San Francisco (Prot. H), UCBC	ABK	High	Yes ¹	\$19.80/SF	2
San Francisco (Prot. G), SF104(f)	TBC	High	Yes ¹	\$44.36/SF	2
San Francisco (Prot. H), SF104(f)	TBC	High	Yes ¹	\$26.90/SF	2
3-storey, 40'x80' Commercial Bldg	ABK	High	UNK	\$53.26/SF	3
3-storey, 40'x80' Commercial Bldg	ABK	Moderate	UNK	\$31.59/SF	3

¹ At the time, San Francisco was considering mandating comprehensive URM retrofitting, but had not done so yet; however, many jurisdictions throughout California had and so familiarity of designers and contractors was

² Unit costs adjusted to 2014 Canadian dollars assumign the following exchange rates as follows: 1987=\$0.75USD, 1990=\$0.85USD, 1996=\$0.73USD. Inflation Index as per Reed Construction Data (2012).

³ Source 1= FEMA (1988), Source 2=Rutherford and Chekene (1990), Source 3=Rutherford and Chekene (1997)

⁴ "ABK" denotes a standard that is based on research by ABK (1984); "TBC" denotes a standard that is based on typical building code provisions for seismic design

The unit costs vary significantly – by almost a factor of 10. Also note that these are simply the mean values. Where reported, upper and lower bounds often were up to 100% higher and lower than the mean value (eg. for a mean of \$20/SF, the lower bound could be \$10/SF and the upper bound could be \$40/SF). Comparing the values to Victoria, the mean value of \$40/SF as well as the minimum (\$14.82/SF) and maximum (\$128/SF) values appear quite consistent. Note that the two highest estimates are for “typical building code” type of design standards. These standards rely on making the structure quite stiff and thus often require much more retrofitting work; “ABK” type standards design the structure to be more flexible (thereby reducing forces) and contain various other provisions to

make use of the existing structure. As such, designs to these standards are typically less costly.

A.5 Cost Estimate Data

As discussed in Section A.2.2, detailed estimates were prepared by the author for various targeted upgrades. The complete estimates are provided on the following pages. See Table A.2 for summary figures.

A.5.1 Parapet Bracing Estimate #1

Parapet Bracing Estimate - Detail #1: Thru-Bolted Anchors

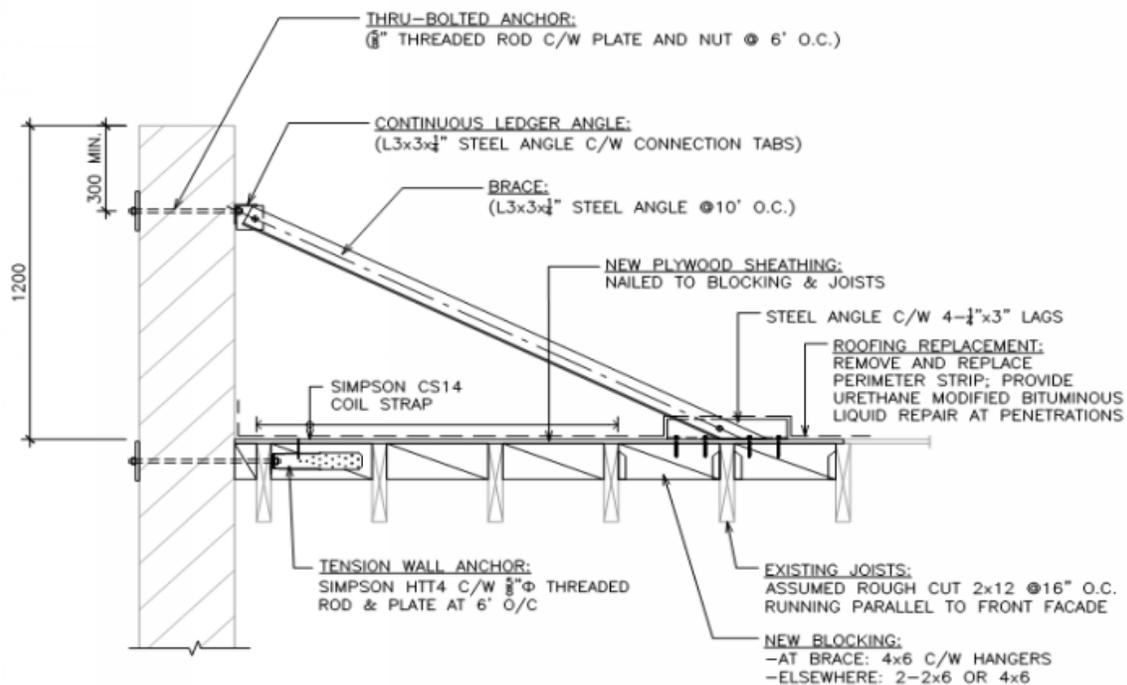
SCOPE OF WORK:

Repoint parapet & provide parapet bracing along a 30' streetfront façade for a 1-3 storey building (i.e. accessible by ladder) as shown in SK-2. Parapet bracing designed to force levels as specified in 1993 NRC Seismic Evaluation Guidelines.

COST SUMMARY

Construction Cost (incl. OH&P)	\$8,216
Soft Costs (15%)	\$1,232
Subtotal=	\$9,448
GST (5%)	\$472.40

Total=	\$9,920.49
Construction Cost/LF=	\$273.86 (Tall Parapets - h/t>1.5:1)
Construction Cost/LF=	\$141.61 (Short Parapets - h/t<1.5:1)
Cost/LF=	\$330.68 (Tall Parapets - h/t>1.5:1)
Cost/LF=	\$171.00 (Short Parapets - h/t<1.5:1)



ITEMIZED ESTIMATE

Item	Div	Cost Code	Item Description	QTY	Unit	OH&P-10%		OH&P-50%		OH&P-10%		Basic Unit Total	Unit Total (O&P+LF)	Total
						Mat'l	(Loc)	Labour	(Loc)	Equip.	(Loc)			
1	4	04 01 20.20 0300	Repointing Inside Face <i>Cut and repoint brick, hard mortar, running bond</i>	120	SF	\$0.53	1.67	\$4.41	0.94	\$0.00	0.94	\$4.94	\$7.19	\$862.21
2	5	05 12 23.40 0474	Steel Angles (Braces & Ledger) <i>L3x2x3/8, Field Fabricated</i>	40.4	LF	\$4.43	1.07	\$18.45	0.87	\$1.87	0.87	\$24.75	\$62.31	\$2,517.33
3	5	05 12 23.65 2100	Steel Plates (6x6x3/8) <i>1/4thk, no shop fabrication</i>	3.0	SF	\$12.84	1.07	\$0.00	0.87	\$0.00	0.87	\$12.84	\$30.31	\$90.93
4	5	05 05 13.50 5900	Galvanize Angles & Plates <i>Galvanizing in shop, under 1 ton</i>	0.25	Ton	\$810.00	1.07	\$0.00	0.87	\$0.00	0.87	\$810.00	\$956.04	\$239.01
5	5	05 05 23.10 2100	Threaded Rod/Bolts <i>5/8" diam, 10" long</i>	12	Ea.	\$9.21	1.07	\$12.00	0.87	\$0.00	0.87	\$21.21	\$26.57	\$318.83
6	6	06 05 05.10 6096	Strip old sheathing <i>Demo, board sheathing from roof</i>	240	SF	\$0.00	0.98	\$0.40	0.85	\$0.00	0.85	\$0.40	\$1.02	\$245.09
7	6	06 16 36.10 0205	New Sheathing <i>Plywood on roof, 5/8" thick, pneumatic nailed</i>	240	SF	\$0.62	0.98	\$0.45	0.85	\$0.00	0.85	\$1.07	\$2.49	\$597.52
8	6	06 05 23.60 7032	Simpson HTT4 Holddowns <i>Tension Ties, 19-1/8" long, 16 ga., 3/4" anchor bolt</i>	6	Ea.	\$20.00	0.98	\$0.20	0.85	\$0.00	0.85	\$20.20	\$21.88	\$131.26
9	6	06 11 10.02 2660	4x6 Blocking <i>Miscellaneous, to wood construction, 2x8</i>	0.05	MBF	\$565.00	0.98	\$1,300.00	0.85	\$0.00	0.85	\$1,865.00	\$2,270.38	\$113.52
10	6	06 05 23.60	Hangers for 4x6 Blocking <i>Joist and beam hangers, 18ga., galv, for 4x6 to 4x10 joist</i>	24	Ea.	\$2.82	0.98	\$2.28	0.85	\$0.00	0.85	\$5.10	\$5.96	\$143.03
11	7		2-ply Roofing <i>Strip & replace</i>	240	SF	\$2.00	1.15	\$4.00	0.89	\$0.00	0.89	\$6.00	\$7.86	\$1,885.39

Subtotal Construction Cost=	\$7,144.11
General Requirements (15%)=	\$1,071.62
Total Construction Cost=	\$8,215.73

A.5.2 Parapet Bracing Estimate #2

Parapet Bracing Estimate - Detail #2: Epoxy Anchors

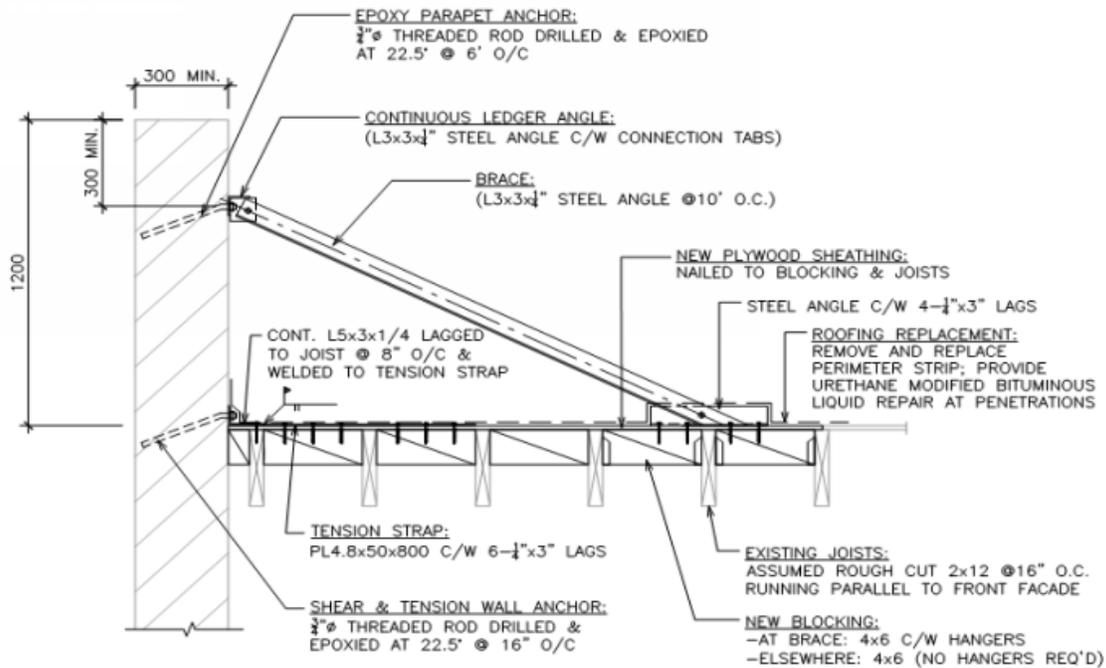
SCOPE OF WORK:

Repoint parapet & provide parapet bracing along a 30' streetfront façade for a 1-3 storey building (i.e. accessible by ladder) as shown. Parapet bracing designed to force levels as specified in 1993 NRC Seismic Evaluation Guidelines. Note that for 2-wythe walls, epoxy anchors are not acceptable.

COST SUMMARY

Construction Cost (incl. OH&P)	\$12,859
Soft Costs (15%)	\$1,929
Subtotal=	\$14,788
GST (5%)	\$739.39
Total=	\$15,527.13

Construction Cost/LF=	\$428.63	(Tall Parapets - h/t>1.5:1)
Construction Cost/LF=	\$280.75	(Short Parapets - h/t<1.5:1)
Total Cost/LF=	\$517.57	(Tall Parapets - h/t>1.5:1)
Total Cost/LF=	\$339.00	(Short Parapets - h/t<1.5:1)



ITEMIZED ESTIMATE

Item	Div	Cost Code	Item Description	QTY	Unit	OH&P=10%		OH&P=50%		OH&P=10%		Basic Unit Total	Unit Total (D&P+LF)	Total
						Mat'l	(Loc)	Labour	(Loc)	Equip.	(Loc)			
1	4	04 01 20.20 0300	Repointing inside face <i>Cut and repoint brick, hard mortar, running bond</i>	120	SF	\$0.53	1.67	\$4.41	0.94	\$0.00	0.94	\$4.94	\$7.19	\$862.21
2	5	05 12 23.40 0474	Steel Angles (Braces & Ledger) <i>L3x2x3/8, Field Fabricated</i>	75.4	LF	\$4.43	1.07	\$18.45	0.87	\$1.87	0.87	\$24.75	\$62.31	\$4,698.18
3	5	05 12 23.65 2100	Steel Plates (6x6x3/8) <i>1/4thk, no shop fabrication</i>	0.0	SF	\$12.84	1.07	\$0.00	0.87	\$0.00	0.87	\$12.84	\$30.31	\$0.00
4	5	05 12 23.65 2100	Steel Strap (2"x32"x3/16) <i>1/4thk, no shop fabrication</i>	1.0	SF	\$6.49	1.07	\$0.00	0.87	\$0.00	0.87	\$6.49	\$15.31	\$15.31
5	5	05 05 21.90 4010	Weld Angle to Strap <i>Field Welding, 1/8" E6011, cost per welder, no engineer</i>	1.0	hr	\$4.22	1.07	\$206.00	0.87	\$15.20	0.87	\$225.42	\$289.01	\$289.01
6	5	05 05 23.30 0020	Lag Screws <i>Steel, 1/4" x 2" long</i>	81	Ea.	\$0.14	1.07	\$2.00	0.87	\$0.00	0.87	\$2.14	\$2.78	\$225.28
7	5	05 05 13.50 5900	Galvanize Angles & Plates <i>Galvanizing in shop, under 1 ton</i>	0.40	Ton	\$810.00	1.07	\$0.00	0.87	\$0.00	0.87	\$810.00	\$956.04	\$382.42
8	5	05 05 23.15 1430	Epoxy Anchors <i>Chemical anchor, w/ rod, 3/4"x9.5" long</i>	29	Ea.	\$20.48	1.07	\$24.00	0.87	\$4.08	0.87	\$48.56	\$59.47	\$1,724.69
9	6	06 05 05.10 6096	Strip old sheathing <i>Demo, board sheathing from roof</i>	240	SF	\$0.00	0.98	\$0.40	0.85	\$0.00	0.85	\$0.40	\$1.02	\$245.09
10	6	06 16 36.10 0205	New Sheathing <i>Plywood on roof, 5/8" thick, pneumatic nailed</i>	240	SF	\$0.62	0.98	\$0.45	0.85	\$0.00	0.85	\$1.07	\$2.49	\$597.52
11	6	06 05 23.60 7032	Simpson HTT4 Holdowns <i>Tension Ties, 19-1/8" long, 16 ga., 3/4" anchor bolt</i>	0	Ea.	\$20.00	0.98	\$0.20	0.85	\$0.00	0.85	\$20.20	\$21.88	\$0.00
12	6	06 11 10.02 2660	4x6 Blocking <i>Miscellaneous, to wood construction, 2x8</i>	0.05	MBF	\$565.00	0.98	\$1,300.00	0.85	\$0.00	0.85	\$1,865.00	\$2,270.38	\$113.52
13	6	06 05 23.60	Hangers for 4x6 Blocking <i>Joist and beam hangers, 18ga., galv, for 4x6 to 4x10 joist</i>	24	Ea.	\$2.82	0.98	\$2.28	0.85	\$0.00	0.85	\$5.10	\$5.96	\$143.03
14	7		2-ply Roofing <i>Strip & replace</i>	240	SF	\$2.00	1.15	\$4.00	0.89	\$0.00	0.89	\$6.00	\$7.86	\$1,885.39

Subtotal Construction Cost=	\$11,181.66
General Requirements (15%)=	\$1,677.25
Total Construction Cost=	\$12,858.90

A.5.3 Veneer Anchor Estimate #1

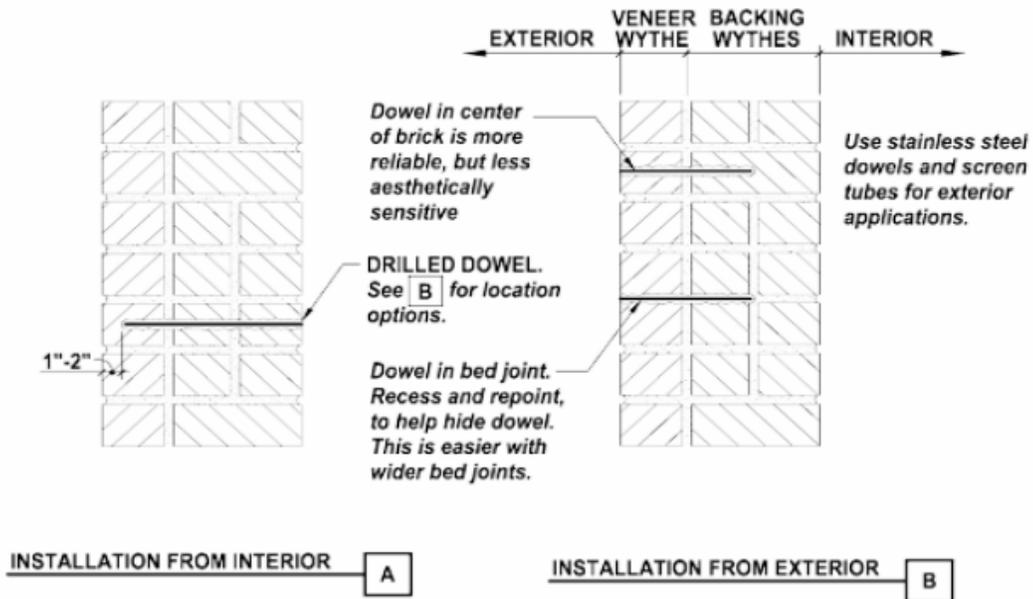
Veneer Anchor Estimate - Detail #1: Epoxied Dowels (From Exterior)

SCOPE OF WORK:

Provide retrofit veneer anchors from the exterior for a 30' long two storey wall (24' high). Anchors are epoxied dowels installed from the exterior, with scaffolding for access. Anchors are assumed to be 3/8" diam. x 8" long, placed at 2'-0" o/c vertically and horizontally.

COST SUMMARY:

Construction Cost (incl. OH&P)	\$10,160
Soft Costs (15%)	\$1,524
Subtotal=	\$11,684
GST (5%)	\$584.21
Total=	\$12,268.49
Construction Cost/SF=	\$14.11
Total Cost/SF=	\$17.04



ITEMIZED ESTIMATE

Item	Div	Cost Code	Item Description	QTY	Unit	Crew	Daily Output	Crew-Days	OH&P=10% Mat'l	(Loc)	OH&P=50% Labour	(Loc)	OH&P=10% Equip.	(Loc)	Basic Unit Total	Unit Total (O&P+LF)	Total
1	5	05 05 23.15 1430	Epoxy Dowels @2'oc Hor. & Vert. Chemical Anchor, w/ rod & epoxy, 3/4" diam. X 9.5" long	180	Ea.	B-89A 1 SW 1 Lab	27	6.66667	\$4.60	1.07	\$20.21	0.87	\$3.35	0.87	\$28.16	\$35.08	\$6,313.65
2	4	04 01 20.20 0300	Spot repoint over dowels Cut & Repoint hard mortar, running bond	27	SF	1 Bric	20	1.35	\$0.53	1.67	\$4.41	0.94	\$0.00	0.94	\$4.94	\$14.37	\$388.00
3	1	01 54 23.70 0090	Scaffolding - Erect & Dismantle Bldg ext wall, no plank, 1-5 stories	7.2	CSF	3 Carp	4	1.8	\$0.00	1.08	\$132.00	1.08	\$0.00	1.08	\$132.00	\$213.84	\$1,539.65
4	1	01 54 23.70 0906	Scaffolding - Mat'l Rental Complete System for face of walls, no plank, rental/mth.	7.2	CSF	--	--	--	\$36.00	1.08	\$0.00	1.08	\$0.00	1.08	\$36.00	\$42.77	\$307.93
5	1	01 54 23.70 5700	Scaff planks - Erect & Dismantle Plans, 2x10x16' up to 50' high	12	Ea.	3 Carp	72	0.16667	\$0.00	1.08	\$14.70	1.08	\$0.00	1.08	\$14.70	\$23.81	\$285.77
Subtotal Construction Cost= \$8,834.99 General Requirements (15%)= \$1,325.25 Total Construction Cost= \$10,160.24																	

A.5.4 Veneer Anchor Estimate #2

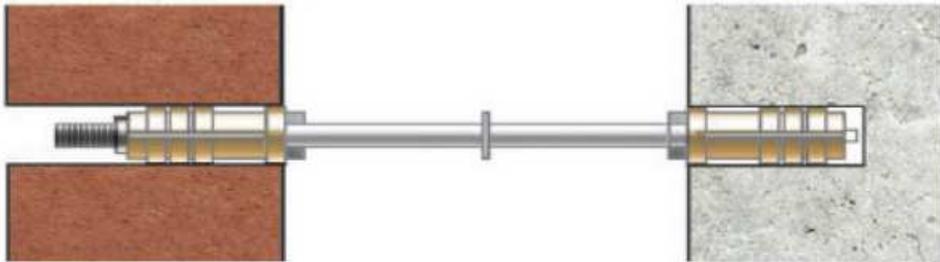
Veneer Anchor Estimate - Detail #2: Expansion Anchors (From Exterior)

SCOPE OF WORK:

Provide retrofit veneer anchors from the exterior for a 30' long two storey wall (24' high). Anchors are mechanical expansion anchors installed from the exterior, with scaffolding for access. Assumed to be placed at 2' o/c vertically and horizontally.

COST SUMMARY:

Construction Cost (incl. OH&P)	\$7,542
Soft Costs (15%)	\$1,131
	Subtotal= \$8,674
GST (5%)	\$433.69
	Total= \$9,107.52
	Construction Cost/SF= \$10.48
	Total Cost/SF= \$12.65



ITEMIZED ESTIMATE

Item	Div	Cost Code	Item Description	QTY	Unit	Crew	Daily Output	Crew-Days	OH&P=10% Mat'l	(Loc)	OH&P=50% Labour	(Loc)	OH&P=10% Equip.	(Loc)	Basic Unit Total	Unit Total (O&P+LF)	Total				
1	5	05 05 23.20 0500	Exp. Anchors @2'oc Hor. & Vert. Wedge anchors, 3/8" diam. X 5" long (no layout & drilling)	180	Ea.	1 Carp	93.333	1.92857	\$6.93	1.07	\$3.15	0.87	\$0.00	0.87	\$10.08	\$12.30	\$2,213.94				
2	3	03 82 16.10 0200	Drill holes for anchors 1/2" diameter, up to 4" (+2")	180	Ea.	1 Carp	175	1.02857	\$0.07	1.51	\$7.42	0.90	\$0.00	0.90	\$7.49	\$10.13	\$1,823.38				
3	4	04 01 20.20 0300	Spot repoint over dowels Cut & Repoint hard mortar, running bond	27	SF	1 Bric	20	1.35	\$0.53	1.67	\$4.41	0.94	\$0.00	0.94	\$4.94	\$14.37	\$388.00				
4	1	01 54 23.70 0090	Scaffolding - Erect & Dismantle Bldg ext wall, no plank, 1-5 stories	7.2	CSF	3 Carp	4	1.8	\$0.00	1.08	\$132.00	1.08	\$0.00	1.08	\$132.00	\$213.84	\$1,539.65				
5	1	01 54 23.70 0906	Scaffolding - Mat'l Rental Complete System for face of walls, no plank, rental/rmth.	7.2	CSF	--	--	--	\$36.00	1.08	\$0.00	1.08	\$0.00	1.08	\$36.00	\$42.77	\$307.93				
6	1	01 54 23.70 5700	Scaff planks - Erect & Dismantle Plans, 2x10x16' up to 50' high	12	Ea.	3 Carp	72	0.16667	\$0.00	1.08	\$14.70	1.08	\$0.00	1.08	\$14.70	\$23.81	\$285.77				
<table border="1"> <tr> <td>Subtotal Construction Cost=</td> <td>\$6,558.66</td> </tr> <tr> <td>General Requirements (15%)=</td> <td>\$983.80</td> </tr> <tr> <td>Total Construction Cost=</td> <td>\$7,542.46</td> </tr> </table>																Subtotal Construction Cost=	\$6,558.66	General Requirements (15%)=	\$983.80	Total Construction Cost=	\$7,542.46
Subtotal Construction Cost=	\$6,558.66																				
General Requirements (15%)=	\$983.80																				
Total Construction Cost=	\$7,542.46																				

A.5.5 Wall Anchor Estimate #1

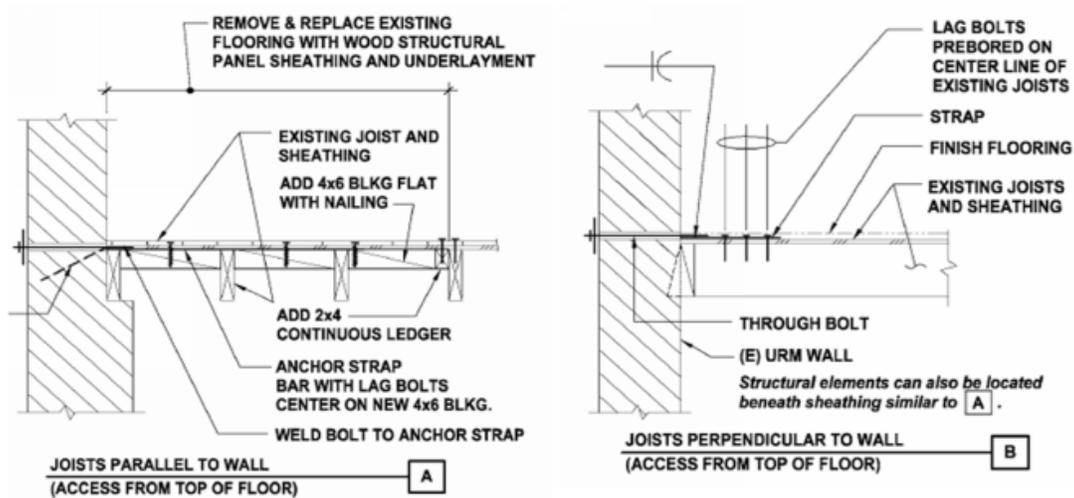
Wall Anchorage Estimate - Detail #1: Thru-Bolted Anchors (Tension Only)

SCOPE OF WORK:

Provide thru-bolted wall tension anchors at 4ft o/c. Provide new sheathing and install new blocking where joists are parallel to the wall. Removal of floor finishes within ± 4 feet of the wall required; existing flooring to be reinstated (assumed carpet/laminate; tile flooring would increase costs significantly)

COST SUMMARY

Construction Cost (incl. OH&P)	\$2,577
Soft Costs (15%)	\$386
Subtotal=	\$2,963
GST (5%)	\$148.15
Total=	\$3,111.21
Construction Cost/LF=	\$85.89
Total Cost/LF=	\$103.71



ITEMIZED ESTIMATE

Item	Div	Cost Code	Item Description	QTY	Unit	O&P=10%		O&P=50%		O&P=10%		Basic Unit Total	Unit Total (O&P+LF)	Total
						Mat'l	(Loc)	Labour	(Lcc)	Equip.	(Loc)			
1	5	05 12 23.65 2100	Steel Plates (6x6x3/8 + 1.5"x48") <i>1/4thk, no shop fabrication</i>	6.0	SF	\$25.68	1.07	\$0.00	0.87	\$0.00	0.87	\$25.68	\$60.62	\$363.72
2	5	05 05 21.90 4010	Weld Anchor to Strap <i>Field Welding, 1/8" EG011, cost per welder, no engineer</i>	1.00	hr	\$4.22	1.07	\$206.00	0.87	\$15.20	0.87	\$225.42	\$289.01	\$289.01
3	5	05 05 23.10 2100	Threaded Rod/Bolts @ 4'o/c <i>5/8" diam, 10" long</i>	8	Ea.	\$9.21	1.07	\$12.00	0.87	\$0.00	0.87	\$21.21	\$26.57	\$212.55
4	6	06 05 05.10 6096	Strip old sheathing <i>Demo, board sheathing from roof</i>	120	SF	\$0.00	0.98	\$0.40	0.85	\$0.00	0.85	\$0.40	\$1.02	\$122.54
5	6	06 16 36.10 C205	New Sheathing <i>Plywood on roof, 5/8" thick, pneumatic nailed</i>	120	SF	\$0.62	0.98	\$0.45	0.85	\$0.00	0.85	\$1.07	\$2.49	\$298.76
6	6	06 05 23.60	Hangers for 4x6 Blocking <i>Joist and beam hangers, 18ga., galv. for 4x6 to 4x10 joist</i>	54	Ea.	\$2.82	0.98	\$2.28	0.85	\$0.00	0.85	\$5.10	\$5.96	\$321.82
7	6	06 11 10.02 2660	4x6 Blocking <i>Miscellaneous, to wood construction, 2x8</i>	0.1	M3F	\$565.00	0.98	\$1,300.00	0.85	\$0.00	0.85	\$1,865.00	\$2,270.38	\$227.04
8	9	09 05 05.20 2000	Remove flooring <i>Vinyl composition tile, 12"x12"</i>	120	SF	\$0.00	1.16	\$0.56	0.85	\$0.00	0.85	\$0.56	\$0.71	\$35.38
9	9	09 09 13.10 C100	Re-instate flooring <i>Average labour cost for installing flooring</i>	120	SF	\$0.00	1.15	\$2.00	0.89		0.89	\$2.00	\$2.66	\$319.68

Subtotal Construction Cost=	\$2,240.50
General Requirements (15%)=	\$336.07
Total Construction Cost=	\$2,576.57

A.5.6 Wall Anchor Estimate #2

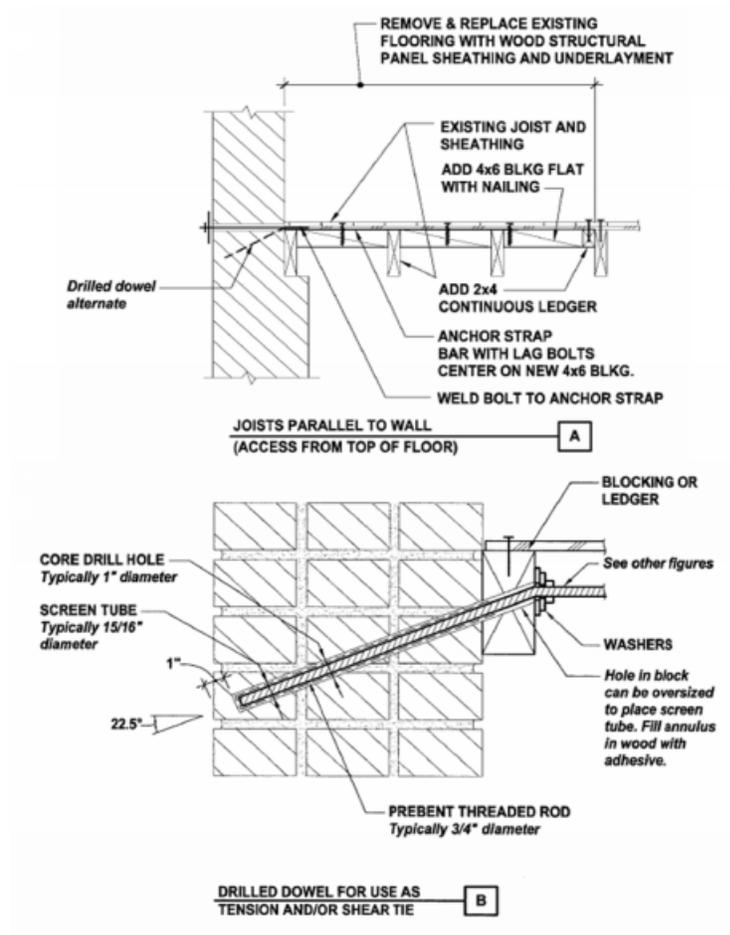
Wall Anchorage Estimate - Detail #2: Epoxy Anchors (Tension Only)

SCOPE OF WORK:

Provide epoxied wall tension anchors at 16" o/c for 30ft storefront. Provide new sheathing and new blocking where joists are parallel to the wall. Removal of floor finishes within +4 feet of the wall; existing flooring to be reinstated (assumed carpet/laminate; tile flooring would increase costs significantly)

COST SUMMARY

Construction Cost (incl. OH&P)	\$3,609
Soft Costs (15%)	\$541
Subtotal=	\$4,150
GST (5%)	\$207.49
Total=	\$4,357.37
Construction Cost/LF=	\$120.29
Total Cost/LF=	\$145.25



ITEMIZED ESTIMATE

Item	Div	Cost Code	Item Description	QTY	Unit	OH&P=10%		OH&P=50%		OH&P=10%		Basic Unit Total	Unit Total (O&P+LF)	Total
						Mat'l	(Loc)	Labour	(Loc)	Equip.	(Loc)			
1	5	05 12 23.65 2100	Steel Plates (8-1.5"x48"long) <i>1/4thk, no shop fabrication</i>	11.00	SF	\$8.65	1.07	\$0.00	0.87	\$0.00	0.87	\$8.65	\$20.42	\$224.61
2	5	05 05 21.90 4010	Weld Anchor to Strap <i>Field Welding, 1/8" E6011, cost per welder, no engineer</i>	1.00	hr	\$4.22	1.07	\$206.00	0.87	\$15.20	0.87	\$225.42	\$289.01	\$289.01
3	5	05 05 23.15 1430	Epoxy Anchors @16" o/c <i>Chemical anchor, w/ rod, 3/4"x9.5" long</i>	22	Ea.	\$20.48	1.07	\$24.00	0.87	\$4.08	0.87	\$48.56	\$46.46	\$1,022.02
4	5	06 05 05.10 6096	Strip old sheathing <i>Demo, board sheathing from roof</i>	120	SF	\$0.00	0.98	\$0.40	0.85	\$0.00	0.85	\$0.40	\$1.02	\$122.54
5	5	06 16 36.10 0205	New Sheathing <i>Plywood on roof, 5/8" thick, pneumatic nailed</i>	120	SF	\$0.62	0.98	\$0.45	0.85	\$0.00	0.85	\$1.07	\$2.49	\$298.76
6	5	06 05 23.60	Hangers for 4x6 Blocking <i>Joist and beam hangers, 18ga., galv, for 4x6 to 4x10 joist</i>	54	Ea.	\$2.82	0.98	\$2.28	0.85	\$0.00	0.85	\$5.10	\$5.96	\$321.82
7	5	06 11 10.02 2660	4x6 Blocking <i>Miscellaneous, to wood construction, 2x8</i>	0.2	MBF	\$565.00	0.98	\$1,300.00	0.85	\$0.00	0.85	\$1,865.00	\$2,270.38	\$454.08
8	9	09 05 05.20 2000	Remove flooring <i>Vinyl composition tile, 12"x12"</i>	120	SF	\$0.00	1.16	\$0.56	0.85	\$0.00	0.85	\$0.56	\$0.71	\$85.38
9	9	09 09 13.10 0100	Re-instate flooring <i>Average labour cost for installing flooring</i>	120	SF	\$0.00	1.15	\$2.00	0.89		0.89	\$2.00	\$2.66	\$319.68
													Subtotal Construction Cost=	\$3,137.90
													General Requirements (15%)=	\$470.69
													Total Construction Cost=	\$3,608.59

Appendix B

Observed Damage Data

B.1 Observed Damage Data

URM damage statistics were obtained for four historical earthquakes, as discussed in Chapter 4. The statistics were collected by others as noted below:

- 1) Whittier Narrows Earthquake: obtained from Wiggins (1994)
- 2) Loma Prieta Earthquake: obtained as raw data from Mr. Bret Lizundia of Rutherford + Chekene engineers (see Lizundia, 1993)
- 3) Northridge Earthquake: obtained from Mr. Bret Lizundia of Rutherford + Chekene engineers (see Rutherford & Chekene, 1997)
- 4) Canterbury Earthquakes; obtained from Dr. Jason Ingham of the University of Auckland (see Ingham & Griffith, 2011a, 2011b)

Some form of additional work by the author was required in all cases (see Chapter 4). Provided on the following pages are the damage probability matrices (DPM's) that were derived using the procedures discussed in Chapter 4.

DAMAGE STATISTICS FROM 1987 WHITTIER NARROWS EARTHQUAKE

ORIGINALLY FROM WIGGINS (1994) IN TERMS OF MMI; NOW CONVERTED TO Sa(1)

RETROFIT: UNRET.

ATC-13 Damage	Geom. Damage Ratio	AVG MMI=6.0 MMI=5.5 to 6.0		AVG MMI=6.5 MMI=6.0 to 6.5		AVG MMI=7.0 MMI=6.5 to 7.0		AVG MMI=7.5 MMI=7.0 to 7.5		AVG MMI=8.0 MMI=7.5 to 8.0	
		AVG Sa(1)=.07 Sa(1)=.05 to .09		AVG Sa(1)=.11 Sa(1)=.09 to .13		AVG Sa(1)=.16 Sa(1)=.13 to .19		AVG Sa(1)=.24 Sa(1)=.19 to .29		AVG Sa(1)=.34 Sa(1)=.29 to .39	
		# Bldgs	% Bldgs								
None (0%)	0.000	9	90.0%	29	87.9%	18	78.3%	213	62.1%	800	64.0%
Slight (0-1%)	0.005	1	10.0%	1	3.0%	0	0.0%	45	13.1%	66	5.3%
Light (1-10%)	0.032	0	0.0%	3	9.1%	1	4.3%	52	15.2%	230	18.4%
Moderate (10-30%)	0.173	0	0.0%	0	0.0%	2	8.7%	29	8.5%	122	9.8%
Heavy (30-60%)	0.424	0	0.0%	0	0.0%	1	4.3%	2	0.6%	26	2.1%
Major (60-100%)	0.775	0	0.0%	0	0.0%	1	4.3%	2	0.6%	6	0.5%
Destroyed (100%)	1.000	0	0.0%	0	0.0%	0	0.0%	0	0.0%	0	0.0%
Total		10	100.0%	33	100.0%	23	100.0%	343	100.0%	1250	100.0%
		MDF= 0.06%		MDF= 0.31%		MDF= 6.86%		MDF= 2.71%		MDF= 3.56%	
		SD= 0.15%		SD= 0.92%		SD= 18.25%		SD= 8.08%		SD= 9.21%	
		SE _{MDF} = 0.05%		SE _{MDF} = 0.16%		SE _{MDF} = 3.81%		SE _{MDF} = 0.44%		SE _{MDF} = 0.26%	

RETROFIT: PART'L

ATC-13 Damage	Geom. Damage Ratio	AVG MMI=6.0 MMI=5.5 to 6.0		AVG MMI=6.5 MMI=6.0 to 6.5		AVG MMI=7.0 MMI=6.5 to 7.0		AVG MMI=7.5 MMI=7.0 to 7.5		AVG MMI=8.0 MMI=7.5 to 8.0	
		AVG Sa(1)=.07 Sa(1)=.05 to .09		AVG Sa(1)=.11 Sa(1)=.09 to .13		AVG Sa(1)=.16 Sa(1)=.13 to .19		AVG Sa(1)=.24 Sa(1)=.19 to .29		AVG Sa(1)=.34 Sa(1)=.29 to .39	
		# Bldgs	% Bldgs								
None (0%)	0.000	1	33.3%	1	50.0%	4	66.7%	39	66.1%	132	54.8%
Slight (0-1%)	0.005	0	0.0%	0	0.0%	0	0.0%	6	10.2%	21	8.7%
Light (1-10%)	0.032	2	66.7%	1	50.0%	1	16.7%	9	15.3%	57	23.7%
Moderate (10-30%)	0.173	0	0.0%	0	0.0%	0	0.0%	5	8.5%	29	12.0%
Heavy (30-60%)	0.424	0	0.0%	0	0.0%	1	16.7%	0	0.0%	2	0.8%
Major (60-100%)	0.775	0	0.0%	0	0.0%	0	0.0%	0	0.0%	0	0.0%
Destroyed (100%)	1.000	0	0.0%	0	0.0%	0	0.0%	0	0.0%	0	0.0%
Total		3	100.0%	2	100.0%	6	100.0%	59	100.0%	241	100.0%
		MDF= 2.11%		MDF= 1.59%		MDF= 7.60%		MDF= 2.01%		MDF= 3.23%	
		SD= 1.82%		SD= 2.23%		SD= 17.11%		SD= 4.83%		SD= 6.57%	
		SE _{MDF} = 1.05%		SE _{MDF} = 1.58%		SE _{MDF} = 6.98%		SE _{MDF} = 0.63%		SE _{MDF} = 0.42%	

RETROFIT: FULL

ATC-13 Damage	Geom. Damage Ratio	AVG MMI=6.0 MMI=5.5 to 6.0		AVG MMI=6.5 MMI=6.0 to 6.5		AVG MMI=7.0 MMI=6.5 to 7.0		AVG MMI=7.5 MMI=7.0 to 7.5		AVG MMI=8.0 MMI=7.5 to 8.0	
		AVG Sa(1)=.07 Sa(1)=.05 to .09		AVG Sa(1)=.11 Sa(1)=.09 to .13		AVG Sa(1)=.16 Sa(1)=.13 to .19		AVG Sa(1)=.24 Sa(1)=.19 to .29		AVG Sa(1)=.34 Sa(1)=.29 to .39	
		# Bldgs	% Bldgs								
None (0%)	0.000	2	100.0%	6	100.0%	9	90.0%	98	83.8%	256	75.3%
Slight (0-1%)	0.005	0	0.0%	0	0.0%	0	0.0%	6	5.1%	11	3.2%
Light (1-10%)	0.032	0	0.0%	0	0.0%	1	10.0%	11	9.4%	53	15.6%
Moderate (10-30%)	0.173	0	0.0%	0	0.0%	0	0.0%	0	0.0%	16	4.7%
Heavy (30-60%)	0.424	0	0.0%	0	0.0%	0	0.0%	2	1.7%	4	1.2%
Major (60-100%)	0.775	0	0.0%	0	0.0%	0	0.0%	0	0.0%	0	0.0%
Destroyed (100%)	1.000	0	0.0%	0	0.0%	0	0.0%	0	0.0%	0	0.0%
Total		2	100.0%	6	100.0%	10	100.0%	117	100.0%	340	100.0%
		MDF= 0.01%		MDF= 0.01%		MDF= 0.33%		MDF= 1.06%		MDF= 1.83%	
		SD= 0.00%		SD= 0.00%		SD= 1.00%		SD= 5.56%		SD= 5.80%	
		SE _{MDF} = 0.00%		SE _{MDF} = 0.00%		SE _{MDF} = 0.32%		SE _{MDF} = 0.51%		SE _{MDF} = 0.31%	

DAMAGE STATISTICS FROM 1989 LOMA PRIETA EARTHQUAKE

ORIGINALLY FROM LIZUNDIA (1993) IN TERMS OF MMI; NOW CONVERTED TO Sa(1)

RETROFIT: UNRET.

ATC-13 Damage	Geom. Damage Ratio	AVG MMI=6.5		AVG MMI=7.5		AVG MMI=8.5		AVG MMI=9.5	
		AVG Sa(1)=.17		AVG Sa(1)=.29		AVG Sa(1)=.52		AVG Sa(1)=.75	
		Sa(1)=.13 to .21		Sa(1)=.22 to .38		Sa(1)=.38 to .69		Sa(1)=.7 to 1.3	
		# Bldgs	% Bldgs						
None (0%)	0.000	409	69.1%	1199	78.2%	82	45.8%	0	0.0%
Slight (0-1%)	0.005	98	16.6%	141	9.2%	20	11.2%	6	22.2%
Light (1-10%)	0.032	53	9.0%	111	7.2%	29	16.2%	14	51.9%
Moderate (10-30%)	0.173	24	4.1%	34	2.2%	12	6.7%	3	11.1%
Heavy (30-60%)	0.424	5	0.8%	40	2.6%	33	18.4%	3	11.1%
Major (60-100%)	0.775	3	0.5%	8	0.5%	2	1.1%	1	3.7%
Destroyed (100%)	1.000	0	0.0%	0	0.0%	1	0.6%	0	0.0%
Total		592	100.0%	1533	100.0%	179	100.0%	27	100.0%
		MDF= 1.83%		MDF= 2.18%		MDF= 10.98%		MDF= 11.26%	
		SD= 7.45%		SD= 9.00%		SD= 18.92%		SD= 18.63%	
		SE _{MDF} = 0.31%		SE _{MDF} = 0.23%		SE _{MDF} = 1.41%		SE _{MDF} = 3.58%	

STOREYS: ALL

RETROFIT: UNRET

PARAPETS: UNBRACED

ATC-13 Damage	Geom. Damage Ratio	AVG Sa(1)=.15		AVG Sa(1)=.20		AVG Sa(1)=.25		AVG Sa(1)=.30		AVG Sa(1)=.35	
		# Bldgs	% Bldgs								
None (0%)	0	44	88.0%	41	59.4%	56	53.8%	54	52.9%	325	60.3%
Slight (0-1%)	0.001	4	8.0%	11	15.9%	18	17.3%	28	27.5%	116	21.5%
Light (1-10%)	0.032	1	2.0%	6	8.7%	15	14.4%	13	12.7%	58	10.8%
Moderate (10-30%)	0.173	1	2.0%	8	11.6%	13	12.5%	6	5.9%	28	5.2%
Heavy (30-60%)	0.424	0	0.0%	2	2.9%	0	0.0%	0	0.0%	5	0.9%
Major (60-100%)	0.775	0	0.0%	1	1.4%	2	1.9%	1	1.0%	7	1.3%
Total		50	100.0%	69	100.0%	104	100.0%	102	100.0%	539	100.0%
		MDF= 0.43%		MDF= 4.66%		MDF= 4.13%		MDF= 2.21%		MDF= 2.67%	
		SD= 2.48%		SD= 12.46%		SD= 11.77%		SD= 8.58%		SD= 10.22%	
		SE _{MDF} = 0.35%		SE _{MDF} = 1.50%		SE _{MDF} = 1.15%		SE _{MDF} = 0.85%		SE _{MDF} = 0.44%	

STOREYS: ALL

RETROFIT: UNRET

PARAPETS: BRACED

ATC-13 Damage	Geom. Damage Ratio	AVG Sa(1)=.10		AVG Sa(1)=.20		AVG Sa(1)=.25		AVG Sa(1)=.30		AVG Sa(1)=.35	
		# Bldgs	% Bldgs								
None (0%)	0	32	62.7%	131	60.1%	7	43.8%	3	100.0%	525	67.7%
Slight (0-1%)	0.001	18	35.3%	32	14.7%	3	18.8%	0	0.0%	153	19.7%
Light (1-10%)	0.032	1	2.0%	31	14.2%	3	18.8%	0	0.0%	61	7.9%
Moderate (10-30%)	0.173	0	0.0%	17	7.8%	1	6.3%	0	0.0%	29	3.7%
Heavy (30-60%)	0.424	0	0.0%	3	1.4%	1	6.3%	0	0.0%	8	1.0%
Major (60-100%)	0.775	0	0.0%	4	1.8%	1	6.3%	0	0.0%	0	0.0%
Total		51	100.0%	218	100.0%	16	100.0%	3	100.0%	776	100.0%
		MDF= 0.10%		MDF= 3.83%		MDF= 9.19%		MDF= 0.01%		MDF= 1.36%	
		SD= 0.44%		SD= 12.07%		SD= 21.26%		SD= 0.00%		SD= 5.36%	
		SE _{MDF} = 0.06%		SE _{MDF} = 0.82%		SE _{MDF} = 5.31%		SE _{MDF} = 0.00%		SE _{MDF} = 0.19%	

DAMAGE STATISTICS FROM 1994 NORTHRIDGE EARTHQUAKE

ORIGINALLY FROM LIZUNDIA (1997) IN TERMS OF MMI; NOW CONVERTED TO $S_a(1)$

RETROFIT: FULLY RETROFITTED

ATC-13 Damage	Geom. Damage Ratio	AVG MMI=5.5		AVG MMI=6.5		AVG MMI=7.5		AVG MMI=8.5	
		AVG $S_a(1)=.05$		AVG $S_a(1)=.10$		AVG $S_a(1)=.19$		AVG $S_a(1)=.36$	
		Sa(1)=.03 to .06		Sa(1)=.06 to .13		Sa(1)=.13 to .25		Sa(1)=.25 to .48	
		# Bldgs	% Bldgs						
None (0%)	0.000	138	97.9%	131	48.5%	2339	45.9%	58	33.0%
Slight (0-1%)	0.005	2	1.4%	133	49.3%	1503	29.5%	58	33.0%
Light (1-10%)	0.032	1	0.7%	5	1.9%	1144	22.5%	44	25.0%
Moderate (10-30%)	0.173	0	0.0%	1	0.4%	70	1.4%	12	6.8%
Heavy (30-60%)	0.424	0	0.0%	0	0.0%	25	0.5%	3	1.7%
Major (60-100%)	0.775	0	0.0%	0	0.0%	8	0.2%	1	0.6%
Destroyed (100%)	1.000	0	0.0%	0	0.0%	3	0.1%	0	0.0%
Total		141	100.0%	270	100.0%	5092	100.0%	176	100.0%
		MDF=	0.04%	MDF=	0.37%	MDF=	1.49%	MDF=	3.30%
		SD=	0.27%	SD=	1.13%	SD=	5.33%	SD=	8.79%
		SE _{MDF} =	0.02%	SE _{MDF} =	0.07%	SE _{MDF} =	0.07%	SE _{MDF} =	0.66%

Appendix B – Observed Damage Data

DAMAGE STATISTICS FROM 2010/2011 CANTERBURY EARTHQUAKES

STOREYS: ALL
 RETROFIT: UNRET
 PARAPETS: UNBRACED

Geom. ATC-13 Damage Damage Ratio	AVG Sa(1)=.20 Sa(1)=.15 to .25		AVG Sa(1)=.30 Sa(1)=.25 to .35		AVG Sa(1)=.40 Sa(1)=.35 to .45		AVG Sa(1)=.85 Sa(1)=.8 to .9		AVG Sa(1)=1.05 Sa(1)=1.0 to 1.1		AVG Sa(1)=1.30 Sa(1)=1.2 to 1.4		
	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	
None/Slight (0-1%)	0.001	4	44.4%	5	25.0%	9	5.5%	0	0.0%	0	0.0%	0	0.0%
Light (1-10%)	0.032	5	55.6%	8	40.0%	81	49.4%	9	10.6%	2	6.7%	2	16.7%
Moderate (10-30%)	0.173	0	0.0%	2	10.0%	58	35.4%	23	27.1%	12	40.0%	3	25.0%
Heavy (30-60%)	0.424	0	0.0%	3	15.0%	7	4.3%	22	25.9%	7	23.3%	4	33.3%
Major (60-100%)	0.775	0	0.0%	2	10.0%	8	4.9%	29	34.1%	6	20.0%	3	25.0%
Destroyed (100%)	1.000	0	0.0%	0	0.0%	1	0.6%	2	2.4%	3	10.0%	0	0.0%
Total		9	100.0%	20	100.0%	164	100.0%	85	100.0%	30	100.0%	12	100.0%
		MDF= 1.80%		MDF= 17.13%		MDF= 13.89%		MDF= 44.78%		MDF= 42.53%		MDF= 38.36%	
		SD= 1.61%		SD= 25.35%		SD= 18.73%		SD= 28.89%		SD= 31.05%		SD= 27.73%	
		SE _{MDF} = 0.54%		SE _{MDF} = 5.67%		SE _{MDF} = 1.46%		SE _{MDF} = 3.13%		SE _{MDF} = 5.67%		SE _{MDF} = 8.00%	

STOREYS: ALL
 RETROFIT: UNRET
 PARAPETS: BRACED

Geom. ATC-13 Damage Damage Ratio	AVG Sa(1)=.20 Sa(1)=.15 to .25		AVG Sa(1)=.30 Sa(1)=.25 to .35		AVG Sa(1)=.40 Sa(1)=.35 to .45		AVG Sa(1)=.85 Sa(1)=.8 to .9		AVG Sa(1)=1.05 Sa(1)=1.0 to 1.1		AVG Sa(1)=1.30 Sa(1)=1.2 to 1.4	
	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs
None/Slight (0-1%)	0.001	0	0	0.0%	3	5.2%	0	0.0%	0	0.0%	0	0.0%
Light (1-10%)	0.032	0	4	100.0%	26	44.8%	8	44.4%	1	12.5%	0	0.0%
Moderate (10-30%)	0.173	0	0	0.0%	29	50.0%	2	11.1%	3	37.5%	2	40.0%
Heavy (30-60%)	0.424	0	0	0.0%	0	0.0%	5	27.8%	3	37.5%	2	40.0%
Major (60-100%)	0.775	0	0	0.0%	0	0.0%	3	16.7%	0	0.0%	1	20.0%
Destroyed (100%)	1.000	0	0	0.0%	0	0.0%	0	0.0%	1	12.5%	0	0.0%
Total		0	4	100.0%	58	100.0%	18	100.0%	8	100.0%	5	100.0%
		MDF=		MDF= 3.16%		MDF= 10.08%		MDF= 28.03%		MDF= 35.30%		MDF= 39.39%
		SD=		SD= 0.00%		SD= 7.33%		SD= 28.22%		SD= 30.10%		SD= 24.71%
		SE _{MDF} =		SE _{MDF} = 0.00%		SE _{MDF} = 0.96%		SE _{MDF} = 6.65%		SE _{MDF} = 10.64%		SE _{MDF} = 11.05%

RETROFITTED (TYPE A vs. A+B)

STOREYS: ALL
 RETROFIT: TYPE A
 PARAPETS: BRACED

Geom. ATC-13 Damage Damage Ratio	AVG Sa(1)=.15 Sa(1)=.1 to .2		AVG Sa(1)=.30 Sa(1)=.25 to .35		AVG Sa(1)=.40 Sa(1)=.35 to .45		AVG Sa(1)=.85 Sa(1)=.8 to .9		AVG Sa(1)=1.05 Sa(1)=1.0 to 1.1		AVG Sa(1)=1.30 Sa(1)=1.2 to 1.4	
	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs
None/Slight (0-1%)	0.001	2	20.0%	0	7	4.9%	0	0.0%	0	0.0%	0	0.0%
Light (1-10%)	0.032	8	80.0%	0	66	45.8%	5	9.4%	5	23.8%	0	0.0%
Moderate (10-30%)	0.173	0	0.0%	0	63	43.8%	21	39.6%	5	23.8%	1	8.3%
Heavy (30-60%)	0.424	0	0.0%	0	4	2.8%	14	26.4%	6	28.6%	4	33.3%
Major (60-100%)	0.775	0	0.0%	0	4	2.8%	9	17.0%	3	14.3%	6	50.0%
Destroyed (100%)	1.000	0	0.0%	0	0	0.0%	4	7.5%	2	9.5%	1	8.3%
Total		10	100.0%	0	144	100.0%	53	100.0%	21	100.0%	12	100.0%
		MDF= 2.55%		MDF=		MDF= 12.36%		MDF= 39.07%		MDF= 37.59%		MDF= 62.65%
		SD= 1.29%		SD=		SD= 14.18%		SD= 29.30%		SD= 32.16%		SD= 24.12%
		SE _{MDF} = 0.41%		SE _{MDF} =		SE _{MDF} = 1.18%		SE _{MDF} = 4.03%		SE _{MDF} = 7.02%		SE _{MDF} = 6.96%

STOREYS: ALL
 RETROFIT: TYPE A+B
 PARAPETS: BRACED

Geom. ATC-13 Damage Damage Ratio	AVG Sa(1)=.20 Sa(1)=.15 to .25		AVG Sa(1)=.30 Sa(1)=.25 to .35		AVG Sa(1)=.40 Sa(1)=.35 to .45		AVG Sa(1)=.85 Sa(1)=.8 to .9		AVG Sa(1)=1.05 Sa(1)=1.0 to 1.1		AVG Sa(1)=1.20 Sa(1)=1.2 to 1.4	
	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs	# Bldgs	% Bldgs
None/Slight (0-1%)	0.001	0	0.0%	0	8	10.0%	0	0.0%	0	0.0%	0	0.0%
Light (1-10%)	0.032	3	100.0%	0	52	65.0%	8	32.0%	7	33.3%	3	33.3%
Moderate (10-30%)	0.173	0	0.0%	0	19	23.8%	11	44.0%	8	38.1%	3	33.3%
Heavy (30-60%)	0.424	0	0.0%	0	1	1.3%	6	24.0%	4	19.0%	3	33.3%
Major (60-100%)	0.775	0	0.0%	0	0	0.0%	0	0.0%	2	9.5%	0	0.0%
Destroyed (100%)	1.000	0	0.0%	0	0	0.0%	0	0.0%	0	0.0%	0	0.0%
Total		3	100.0%	0	80	100.0%	25	100.0%	21	100.0%	9	100.0%
		MDF= 3.16%		MDF=		MDF= 6.71%		MDF= 18.82%		MDF= 23.11%		MDF= 20.97%
		SD= 0.00%		SD=		SD= 7.48%		SD= 14.90%		SD= 22.86%		SD= 17.22%
		SE _{MDF} = 0.00%		SE _{MDF} =		SE _{MDF} = 0.84%		SE _{MDF} = 2.98%		SE _{MDF} = 4.99%		SE _{MDF} = 5.74%

Appendix C

Fragility Curves

Structural fragility curves were generated based upon observed damage data from past earthquakes, as discussed in Chapter 4. As discussed in Section 4.9.3, upper and lower bound estimates were also developed for use in the sensitivity analysis.

All fragility curves are in terms of HAZUS damage states. Like the default HAZUS curves, the fragility curves are in the form of lognormal cumulative probability distribution functions. The controlling parameters

C.1 Base Structural Fragility Curves

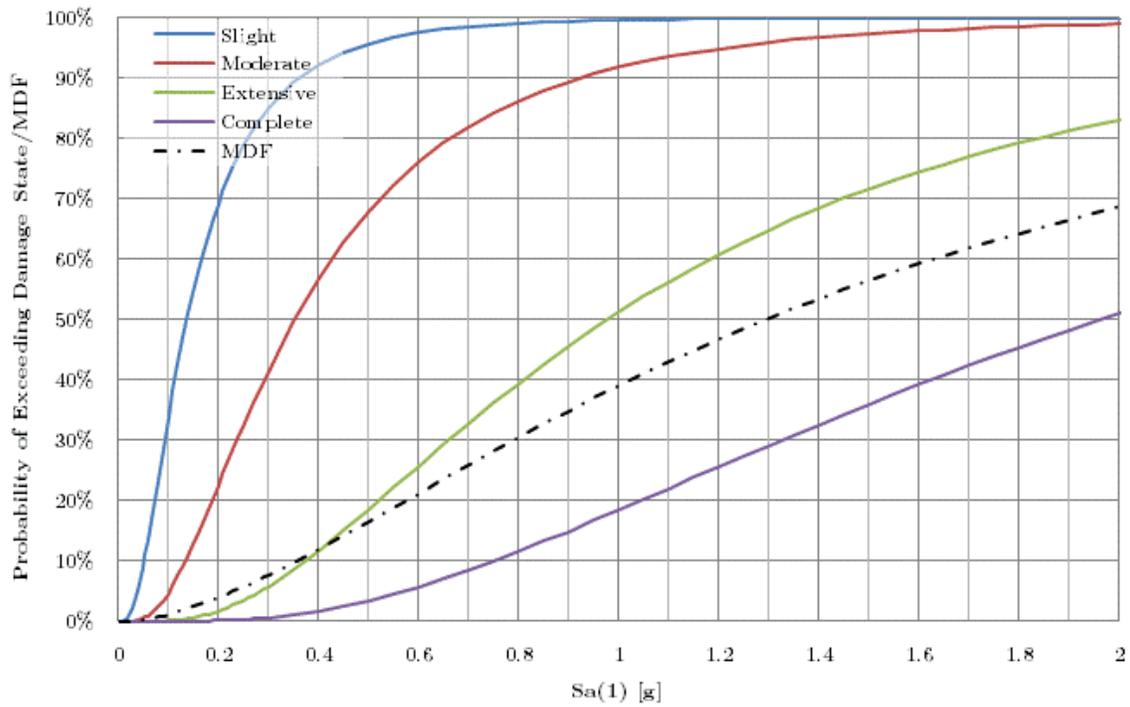
The Base curves (see Section 4.9.3) were the primary focus of the cost-benefit analysis. One such curve exists for each of the four strengthening levels (see Chapter 4).

Unretrofitted Structural Fragility (Base)

Table C.1 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.14	0.75
Moderate	0.35	0.75
Extensive	0.97	0.75
Complete	2.00	0.75

HAZUS Structural Fragility for Vic-Unret (Base)

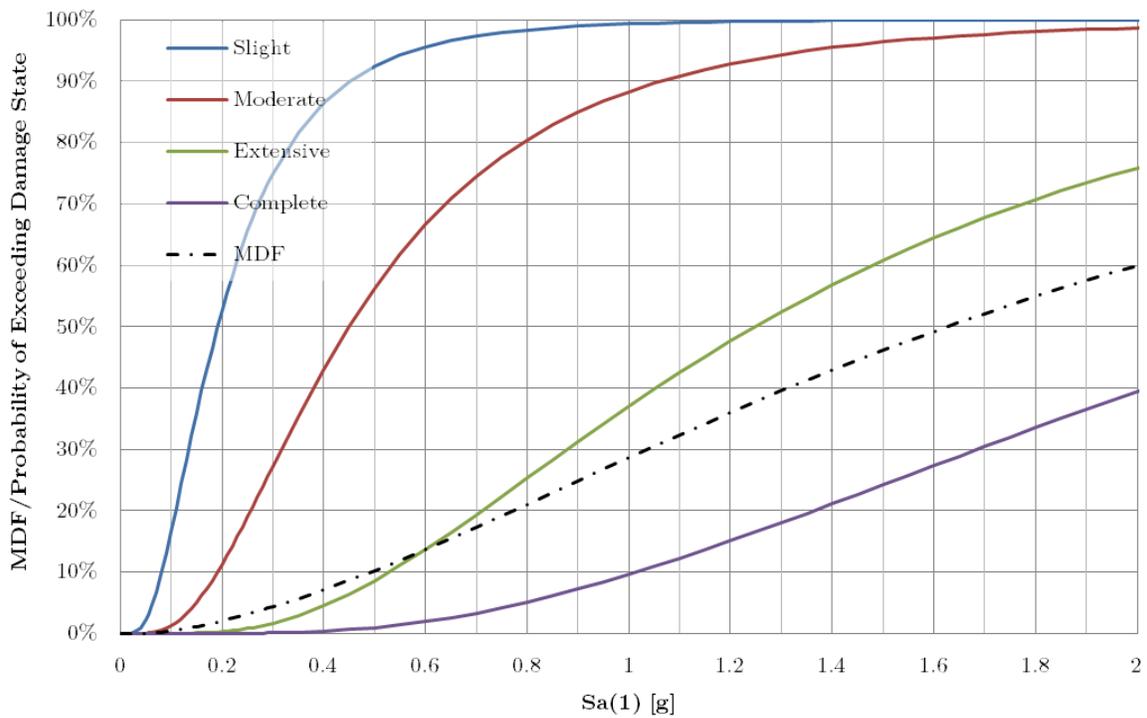


Braced-Parapet Structural Fragility (Base)

Table C.2 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.19	0.67
Moderate	0.45	0.67
Extensive	1.25	0.67
Complete	2.38	0.67

HAZUS Structural Fragility for Vic-Braced (Base)

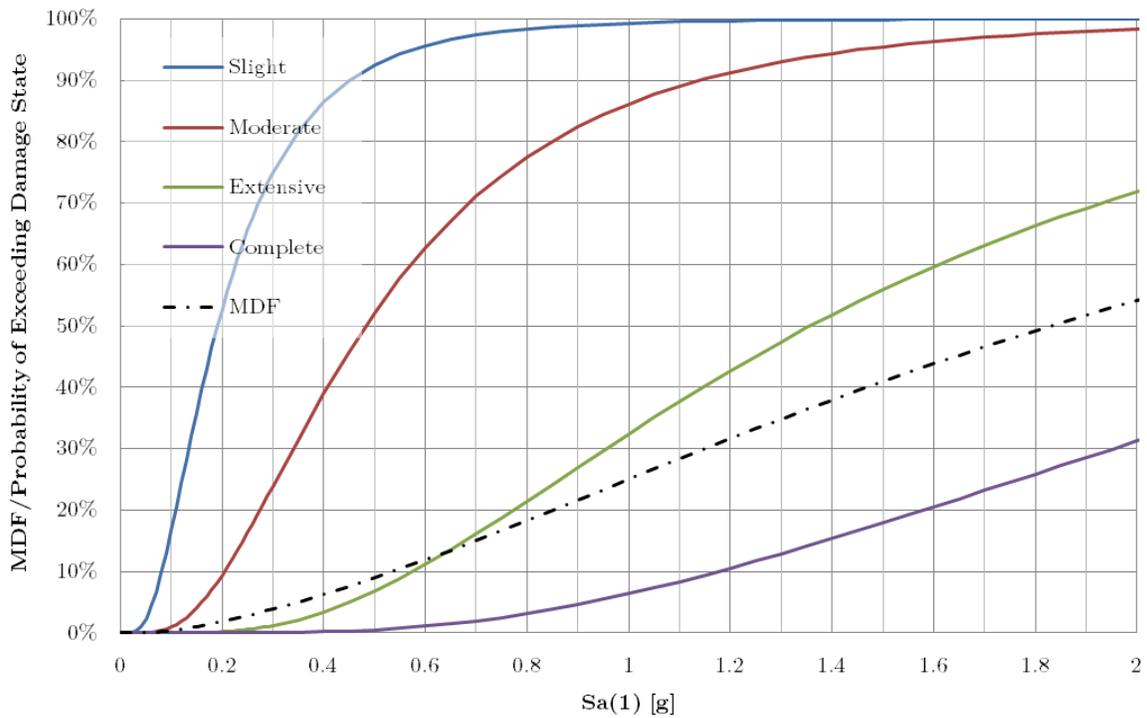


Partially Retrofitted Structural Fragility (Base)

Table C.3 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.19	0.67
Moderate	0.48	0.67
Extensive	1.36	0.67
Complete	2.78	0.67

HAZUS Structural Fragility for Vic-Partial (Base)

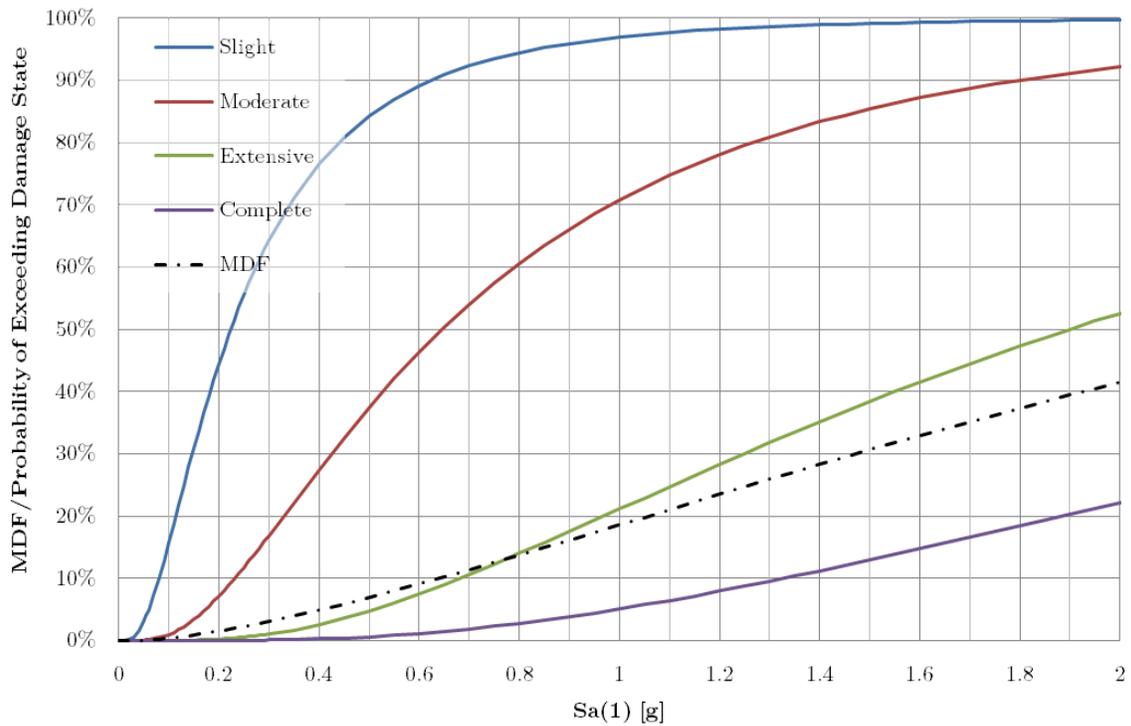


Fully Retrofitted Structural Fragility (Base)

Table C.4 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.22	0.80
Moderate	0.65	0.80
Extensive	1.90	0.80
Complete	3.75	0.80

HAZUS Structural Fragility for Vic-Retrofit (Base)



C.2 Upper Bound Structural Fragility Curve

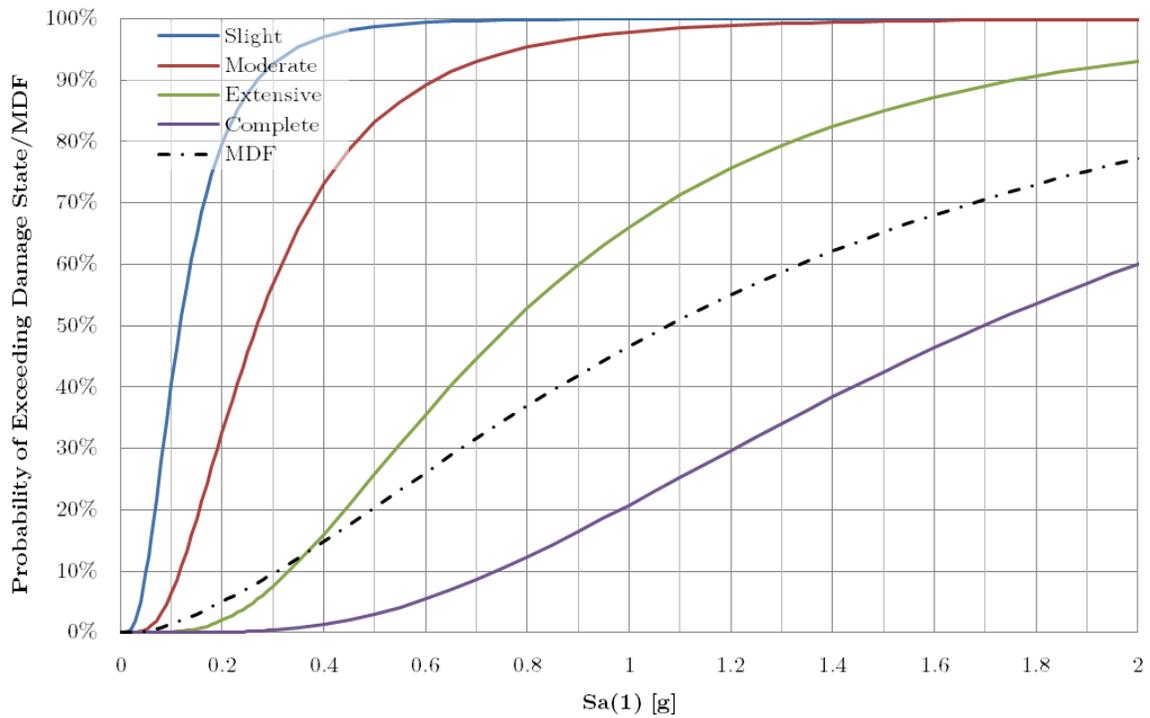
An upper bound (i.e. higher vulnerability) curve for unretrofitted buildings was defined for use in the sensitivity analysis, as discussed in Chapter 5. As discussed in Chapter 4, the upper bound curve was defined as equal to the Canterbury results (i.e. 100% Canterbury/0% California). Such curves could have been defined for each of the strengthening levels, but it was decided not to do so, as their use in the sensitivity analysis would only have diminished the impacts (eg. using ‘upper bound’ curves for both unretrofitted and retrofitted would mask part of the sensitivity).

Unretrofitted Structural Fragility (Upper Bound)

Table C.5 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.12	0.65
Moderate	0.27	0.65
Extensive	0.77	0.65
Complete	1.70	0.65

HAZUS Structural Fragility for Vic-Unret (Upper)



C.3 Lower Bound Structural Fragility Curves

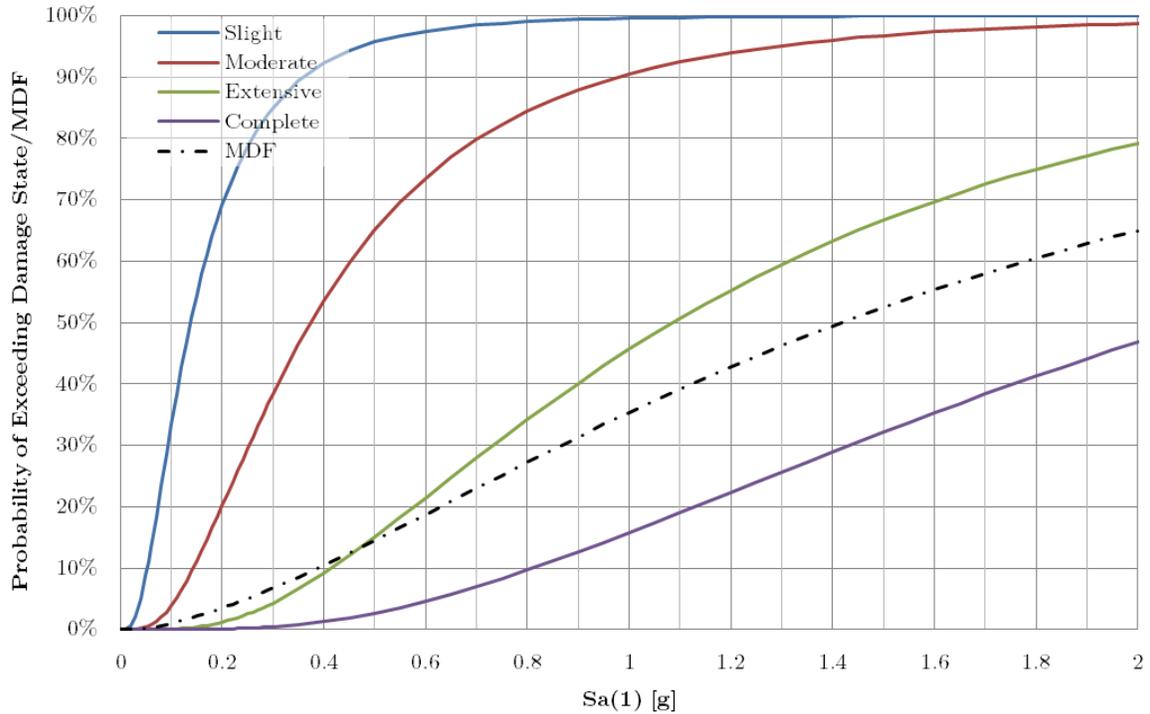
An upper bound (i.e. lower vulnerability) curve for unretrofitted buildings was defined for use in the sensitivity analysis, as discussed in Chapter 5. As discussed in Chapter 4, the upper bound curve was defined as equal to the Canterbury results (i.e. 100% Canterbury/0% California). One such curve exists for each of the four strengthening levels.

Unretrofitted Structural Fragility (Lower Bound)

Table C.6 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.14	0.75
Moderate	0.38	0.75
Extensive	1.10	0.75
Complete	2.15	0.75

HAZUS Structural Fragility for Vic-Unret (Lower)



C.4 Drift-Sensitive Nonstructural Components Fragility Curves

Fragility curves for drift-sensitive non-structural components were defined using default data from HAZUS. As discussed in Chapter 5, the relationships were assigned depending upon the strengthening status as follows:

- Unretrofitted: URMLR (Precode)
- Parapets-Braced & Partially Retrofitted: RM1L (Low Code)
- Fully Retrofitted: RM1L (Moderate Code, Essential Facility)

Fragility curves for the three relationships noted above are provided on the following pages.

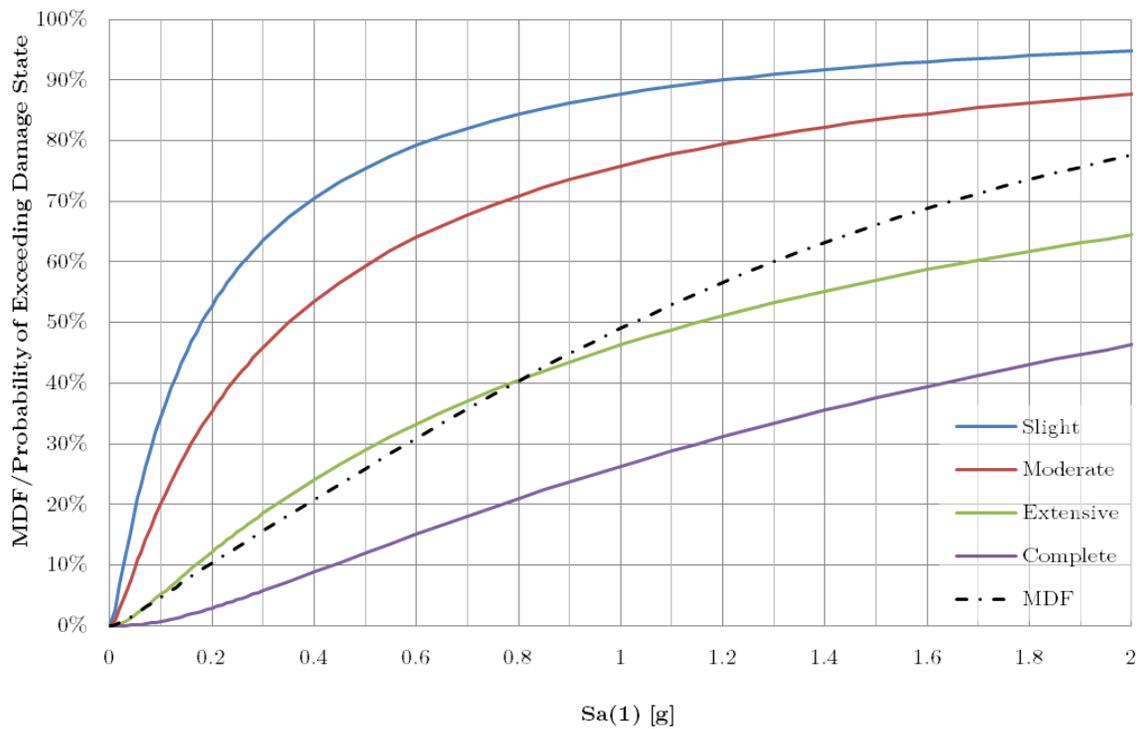
It should be noted that the HAZUS relationships presented here have been converted to $S_a(1\text{sec})$ from $S_d(0.35\text{sec})$. The conversion was based on the spectral shape for Site Class C in Victoria and standard dynamics relationships between pseudoacceleration and displacement.

URMLR (Precode) Drift-Sensitive NSC Fragility

Table C.7 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.18	1.48
Moderate	0.35	1.50
Extensive	1.15	1.50
Complete	2.25	1.28

HAZUS Drift NSC Fragility Curves for URMLR

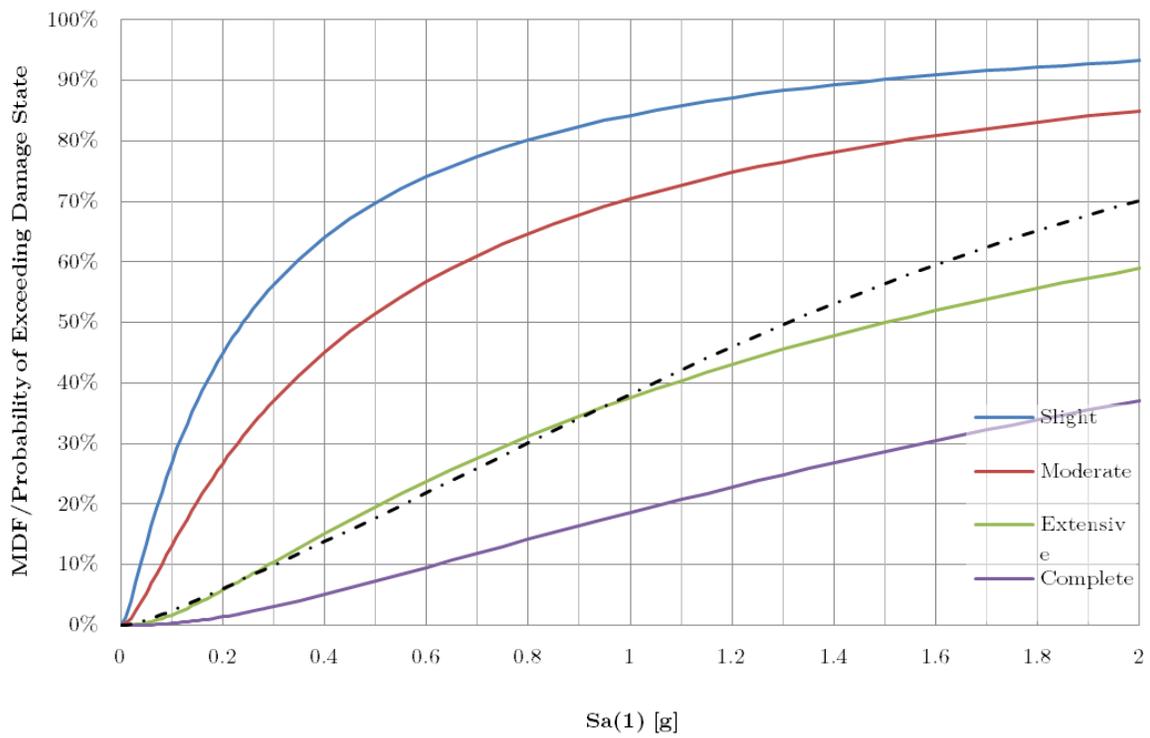


RM1L (Low Code) Drift-Sensitive NSC Fragility

Table C.8 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.24	1.42
Moderate	0.47	1.39
Extensive	1.50	1.28
Complete	3.00	1.23

HAZUS Drift NSC Fragility Curves for RM1L (Low Code)

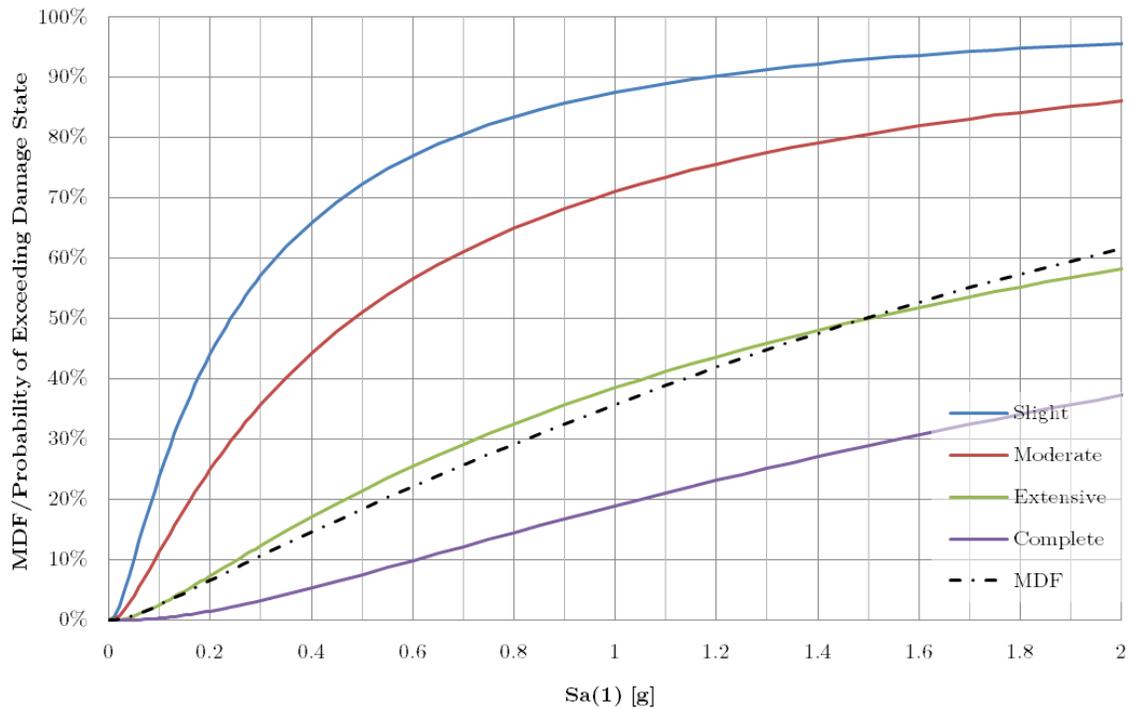


RM1L (Moderate Code, Essential Facility) Drift-Sensitive NSC Fragility

Table C.9 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.24	1.24
Moderate	0.49	1.31
Extensive	1.50	1.39
Complete	3.00	1.25

HAZUS Drift NSC Fragility Curves for RM1L (Mod. Code)



C.5 Acceleration-Sensitive Nonstructural Components Fragility Curves

Fragility curves for acceleration-sensitive non-structural components were defined using default data from HAZUS. As discussed in Chapter 5, the relationships were assigned depending upon the strengthening status as follows:

- Unretrofitted & Parapets-Braced: URMLR (Precode)
- Partially Retrofitted: RM1L (Avg. of Precode and Moderate Code)
- Fully Retrofitted: RM1L (High Code)

Fragility curves for the three relationships noted above are provided on the following pages.

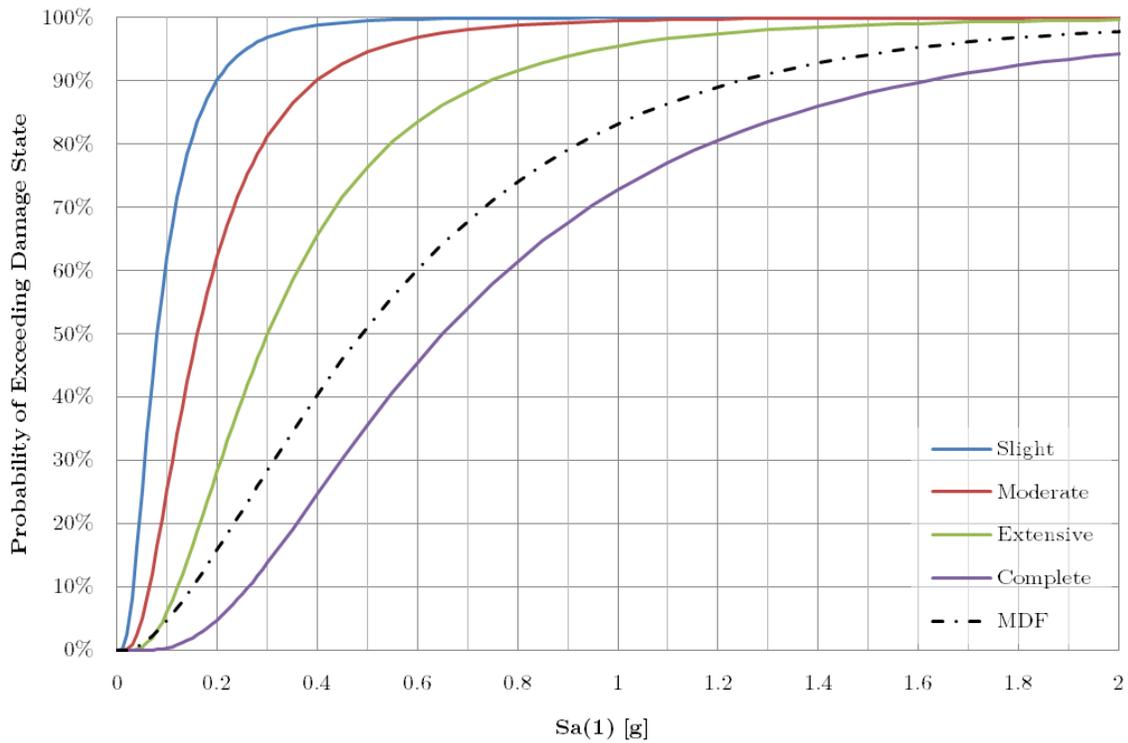
It should be noted that the HAZUS relationships presented here have been converted to $S_a(1\text{sec})$ from $S_a(0.35\text{sec})$. The conversion was based on the spectral shape for Site Class C in Victoria.

URMLR (Pre-Code) Acceleration-Sensitive NSC Fragility

Table C.10 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.08	0.71
Moderate	0.16	0.71
Extensive	0.30	0.71
Complete	0.65	0.71

HAZUS Accel NSC Fragility Curves for URMLR

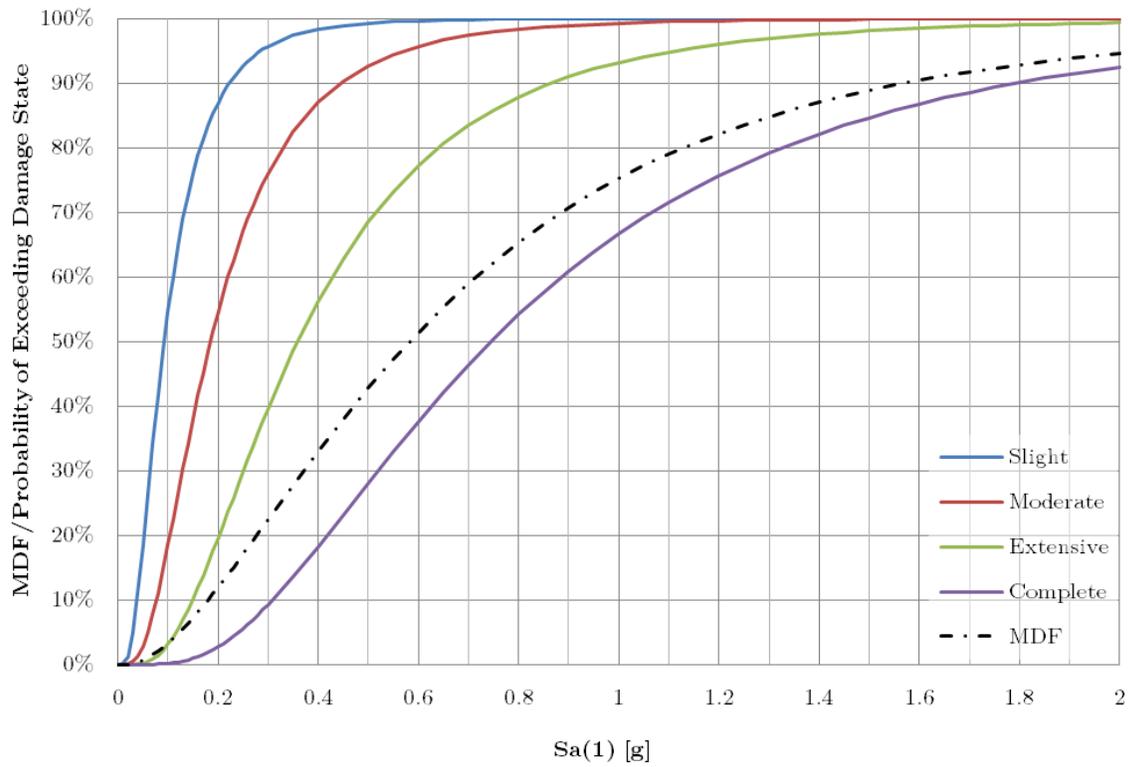


RM1L (Avg. Pre-Code/Moderate Code) Accel.-Sensitive NSC Fragility

Table C.11 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.09	0.69
Moderate	0.19	0.69
Extensive	0.36	0.69
Complete	0.75	0.69

HAZUS Accel NSC Fragility Curves for RM1L (Pre/Mod)

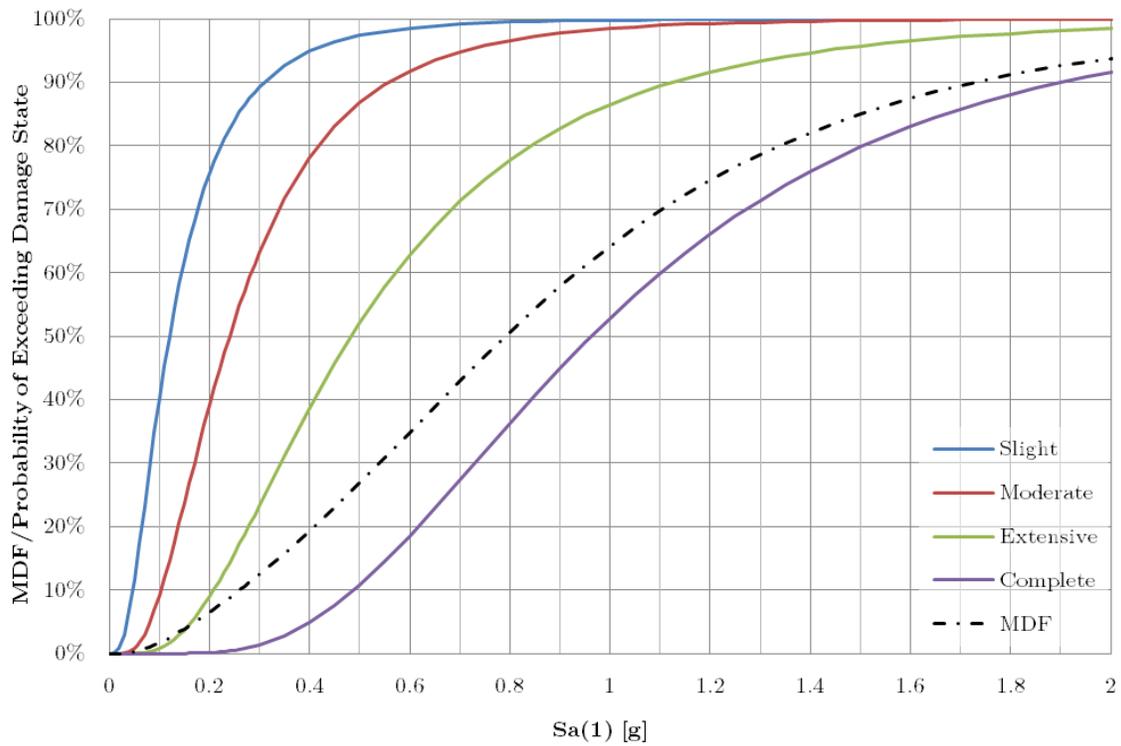


RM1L (High Code) Acceleration-Sensitive NSC Fragility

Table C.12 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.12	0.80
Moderate	0.24	0.80
Extensive	0.48	0.80
Complete	0.97	0.80

HAZUS Accel NSC Fragility Curves for RM1I (High Code)



C.6 Buildings Contents Fragility Curves

Fragility curves for building contents were defined using default data from HAZUS. HAZUS assumes all building contents are acceleration-sensitive and so the same fragilities from the previous section are applied. However, the resulting loss values corresponding to each damage state are halved, as it is assumed that some contents will be salvaged. Thus, the distribution parameters are the same, but the resulting MDF is reduced. As discussed in Chapter 5, the relationships were assigned depending upon the strengthening status as follows:

- Unretrofitted & Parapets-Braced: URMLR (Precode)
- Partially Retrofitted: RM1L (Avg. of Precode and Moderate Code)
- Fully Retrofitted: RM1L (High Code)

Fragility curves for the three relationships noted above are provided on the following pages.

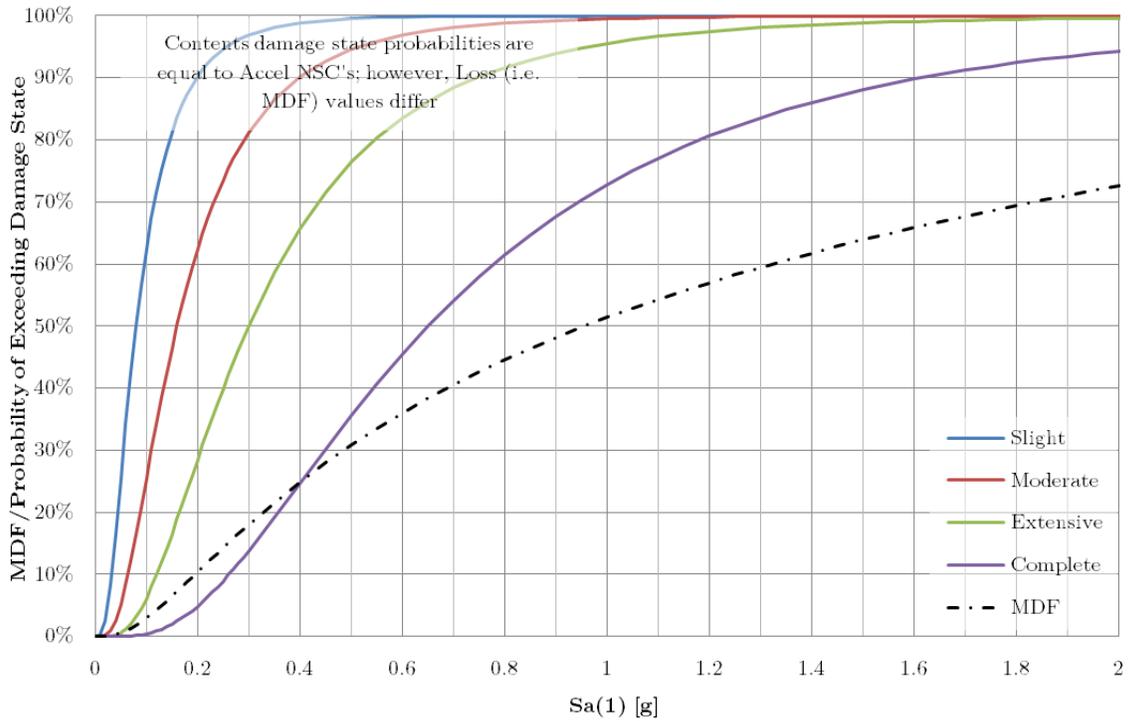
It should be noted that the HAZUS relationships presented here have been converted to $S_a(1\text{sec})$ from $S_a(0.35\text{sec})$. The conversion was based on the spectral shape for Site Class C in Victoria.

URMLR (Precode) Contents Fragility

Table C.13 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.08	0.71
Moderate	0.16	0.71
Extensive	0.30	0.71
Complete	0.65	0.71

HAZUS Contents Fragility Curves for URMLR

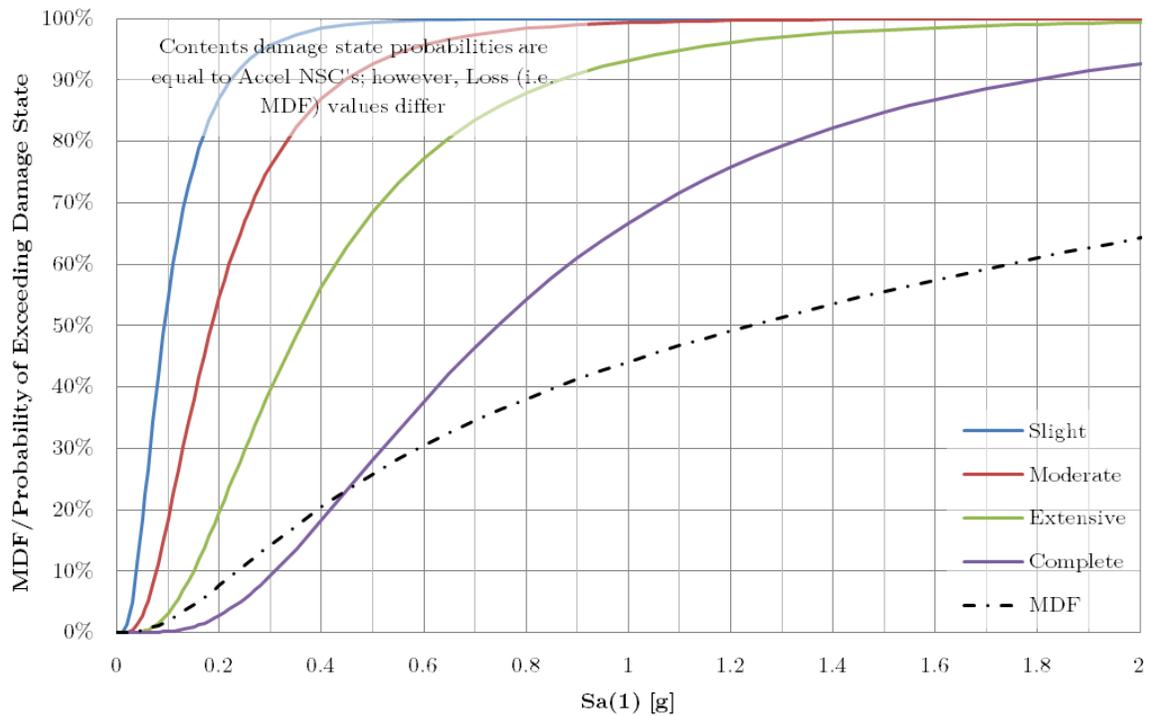


RM1L (Avg. Pre-Code/Moderate Code) Contents Fragility

Table C.14 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.09	0.69
Moderate	0.19	0.69
Extensive	0.36	0.69
Complete	0.75	0.69

HAZUS Contents Fragility Curves for RM1L (Pre/Mod)

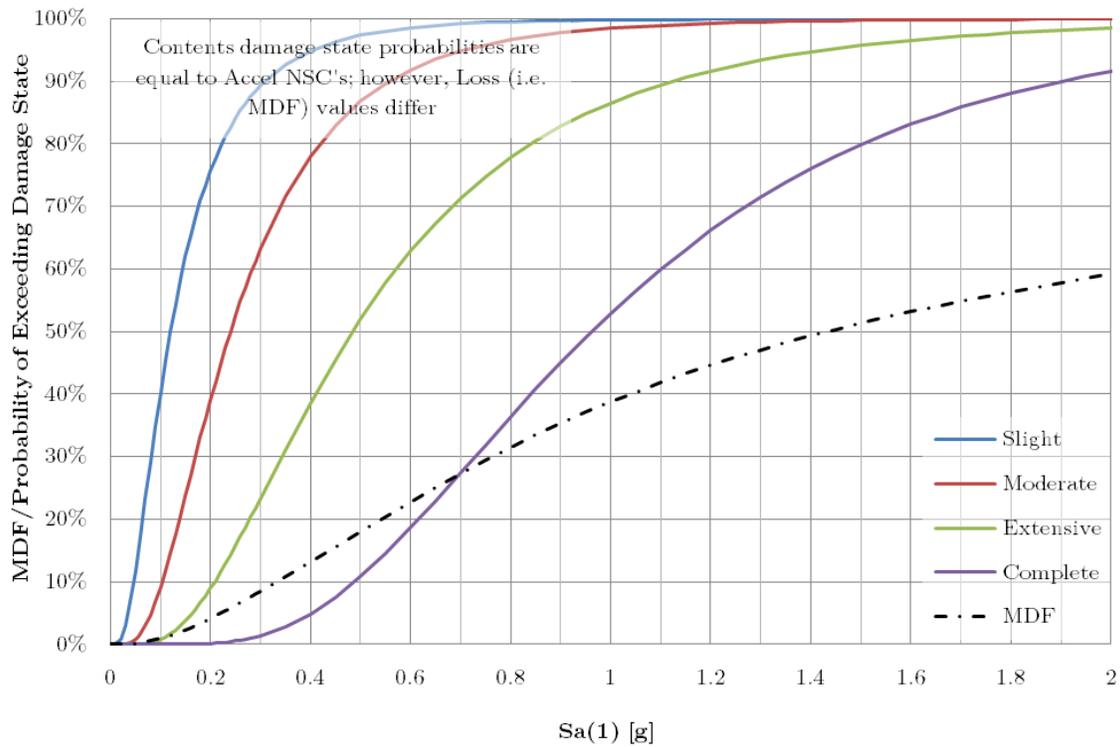


RM1L (High Code) Contents Fragility

Table C.15 – Distribution Parameters

Damage State	Median $S_a(1)$ [g]	Logstandard Deviation (β) $S_a(1)$ [log g]
Slight	0.12	0.80
Moderate	0.24	0.80
Extensive	0.48	0.80
Complete	0.97	0.80

HAZUS Contents Fragility Curves for RM1L (High Code)



Appendix D

Seismic Hazard Data

Seismic hazard data was used in the cost-benefit analysis to derive the expected seismic losses. Hazard values were generated for Victoria, BC using computer program EZ-Frisk, by Risk Management Solutions. Because of the nature of the study, only the results for spectral acceleration at a fundamental period of $T=1\text{sec}$ was of interest. The model used in EZ-Frisk was similar to that for the National Building Code of Canada (developed by the Geological Survey of Canada). The model parameters used were based on GSC's Open File 4459 (OF4459), which is the basis for The 2010 National Building Code of Canada. Unlike OF4459, however, the hazard contribution from a Cascadia subduction event was included in the aggregate seismic hazard. This is essentially consistent with the 2015 National Building Code of Canada (NBCC), with minor differences in model parameters. Ultimately, the calculated model results were found to be within approximately 10% of the published values for the 2015 NBCC at a probability of exceedance of 2% in 50 years; the calculated results were then scaled (up) by approximately 10% to match the published values. Note that the NBCC 2015 values were not directly used because a complete Open File from the GSC had yet to be published at the time of this study, nor had the 2015 NBCC been published. At the same time, however, it was felt that using the 2010 NBCC hazard values would be inappropriate, since these do not include the hazard contribution from a Cascadia subduction event.

Hazard values were calculated for the reference ground condition, Site Class C, and adjusted to represent other soils condition using the long period Foundation Factors (F_v) contained in the 2010 NBCC. The hazard values for $S_a(T=1\text{sec}, \xi=5\%)$ and the foundation factors are presented on the following pages.

D.1 Site Class B

The resulting hazard values for Site Class B are shown in Table D.1 and Figure D.1.

Table D.1 – Seismic Hazard Data (Victoria, Site Class B)

Annual Probability of Exceedance	$S_a(1)$ for RGC (Site Class C) [g]	F_v	$F_v * S_a(1)$
0.150400	0.01	0.6	0.01
0.082400	0.02	0.6	0.01
0.031510	0.06	0.6	0.03
0.020510	0.08	0.6	0.05
0.012370	0.11	0.6	0.07
0.003974	0.22	0.7	0.16
0.001822	0.34	0.7	0.23
0.000975	0.45	0.8	0.36
0.000571	0.56	0.8	0.45
0.000403	0.67	0.8	0.54
0.000234	0.78	0.8	0.63
0.000183	0.89	0.8	0.71
0.000132	1.01	0.8	0.80
0.000082	1.12	0.8	0.89
0.000066	1.23	0.8	0.98
0.000051	1.34	0.8	1.07
0.000035	1.45	0.8	1.16
0.000028	1.56	0.8	1.25
0.000021	1.68	0.8	1.34
0.000018	1.79	0.8	1.43
0.000016	1.90	0.8	1.52
0.000013	2.01	0.8	1.61
0.000011	2.12	0.8	1.70
0.000008	2.23	0.8	1.79

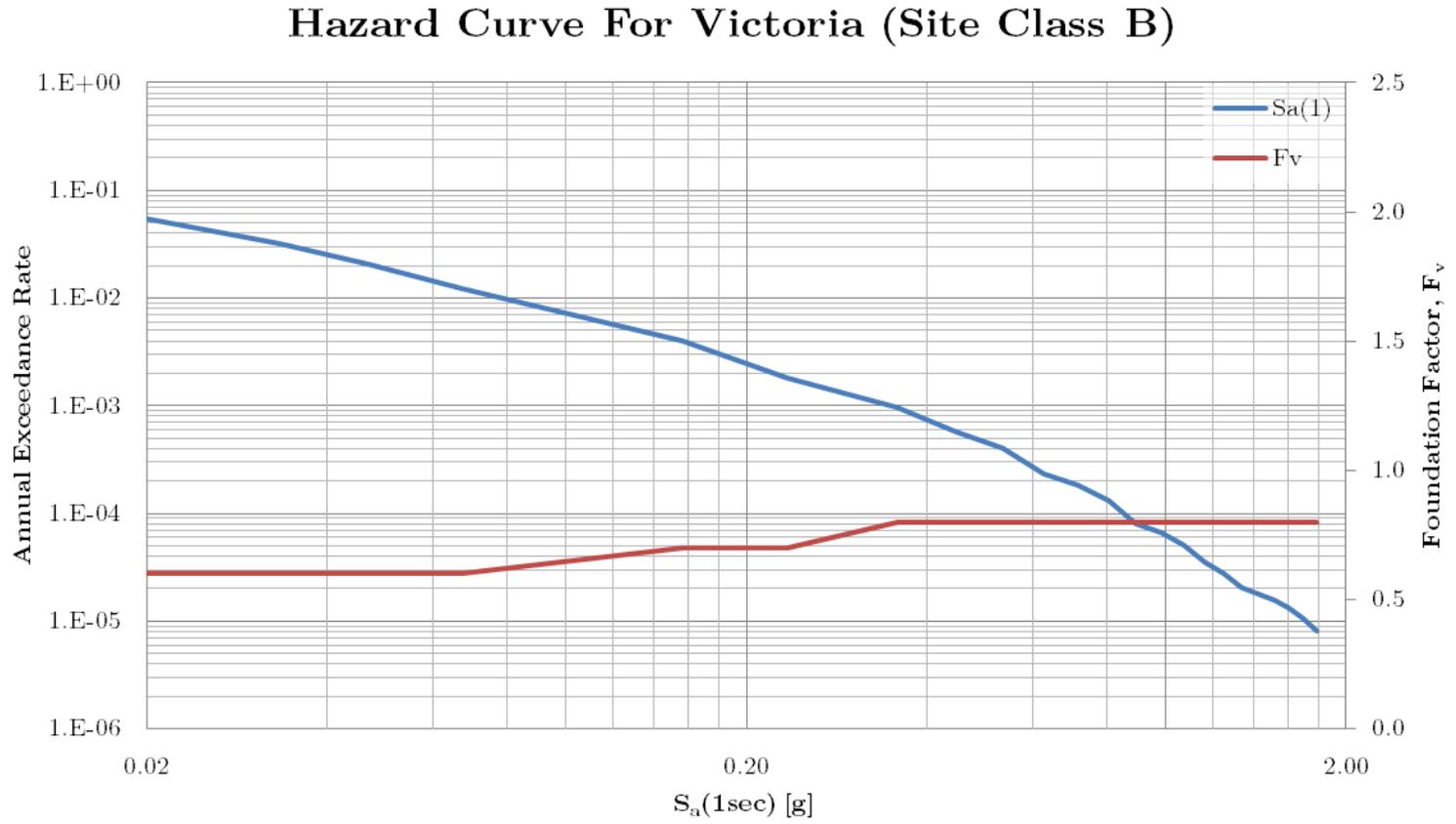


Figure D.1 – Seismic Hazard Data (Victoria, Site Class B)

D.2 Site Class C

The resulting hazard values for Site Class C are shown in Table D.2 and Figure D.2.

Table D.2 – Seismic Hazard Data (Victoria, Site Class C)

Annual Probability of Exceedance	$S_a(1)$ for RGC (Site Class C) [g]	F_v	$F_v * S_a(1)$
0.150400	0.01	1.0	0.01
0.082400	0.02	1.0	0.02
0.031510	0.06	1.0	0.06
0.020510	0.08	1.0	0.08
0.012370	0.11	1.0	0.11
0.003974	0.22	1.0	0.22
0.001822	0.34	1.0	0.34
0.000975	0.45	1.0	0.45
0.000571	0.56	1.0	0.56
0.000403	0.67	1.0	0.67
0.000234	0.78	1.0	0.78
0.000183	0.89	1.0	0.89
0.000132	1.01	1.0	1.01
0.000082	1.12	1.0	1.12
0.000066	1.23	1.0	1.23
0.000051	1.34	1.0	1.34
0.000035	1.45	1.0	1.45
0.000028	1.56	1.0	1.56
0.000021	1.68	1.0	1.68
0.000018	1.79	1.0	1.79
0.000016	1.90	1.0	1.90
0.000013	2.01	1.0	2.01
0.000011	2.12	1.0	2.12
0.000008	2.23	1.0	2.23

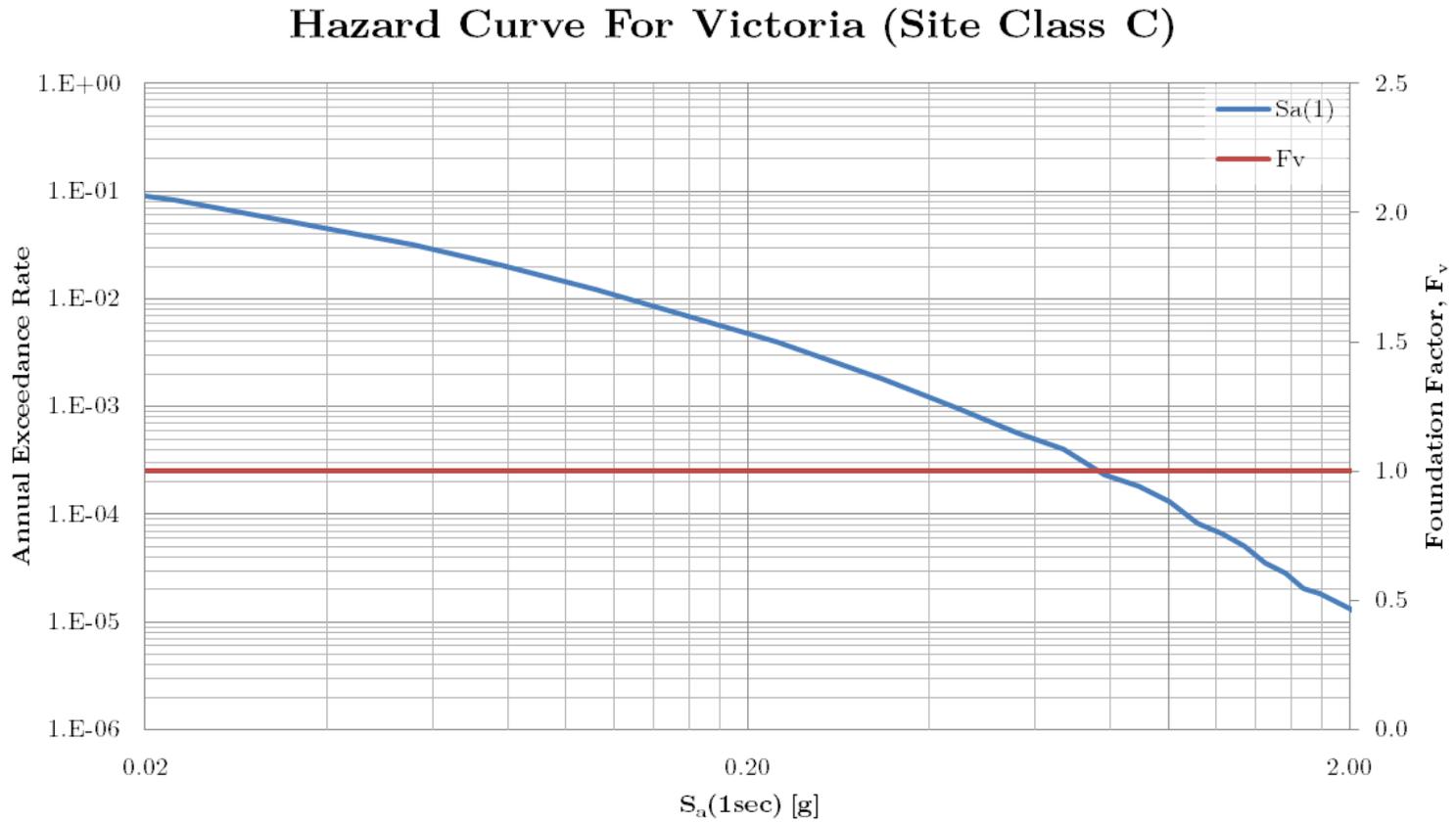


Figure D.2 – Seismic Hazard Data (Victoria, Site Class C)

D.3 Site Class D

The resulting hazard values for Site Class D are shown in Table D.3 and Figure D.3.

Table D.3 – Seismic Hazard Data (Victoria, Site Class D)

Annual Probability of Exceedance	$S_a(1)$ for RGC (Site Class C) [g]	F_v	$F_v * S_a(1)$
0.150400	0.01	1.4	0.01
0.082400	0.02	1.4	0.03
0.031510	0.06	1.4	0.08
0.020510	0.08	1.4	0.11
0.012370	0.11	1.4	0.16
0.003974	0.22	1.3	0.29
0.001822	0.34	1.2	0.40
0.000975	0.45	1.1	0.49
0.000571	0.56	1.1	0.61
0.000403	0.67	1.1	0.74
0.000234	0.78	1.1	0.86
0.000183	0.89	1.1	0.98
0.000132	1.01	1.1	1.11
0.000082	1.12	1.1	1.23
0.000066	1.23	1.1	1.35
0.000051	1.34	1.1	1.47
0.000035	1.45	1.1	1.60
0.000028	1.56	1.1	1.72
0.000021	1.68	1.1	1.84
0.000018	1.79	1.1	1.97
0.000016	1.90	1.1	2.09
0.000013	2.01	1.1	2.21
0.000011	2.12	1.1	2.33
0.000008	2.23	1.1	2.46

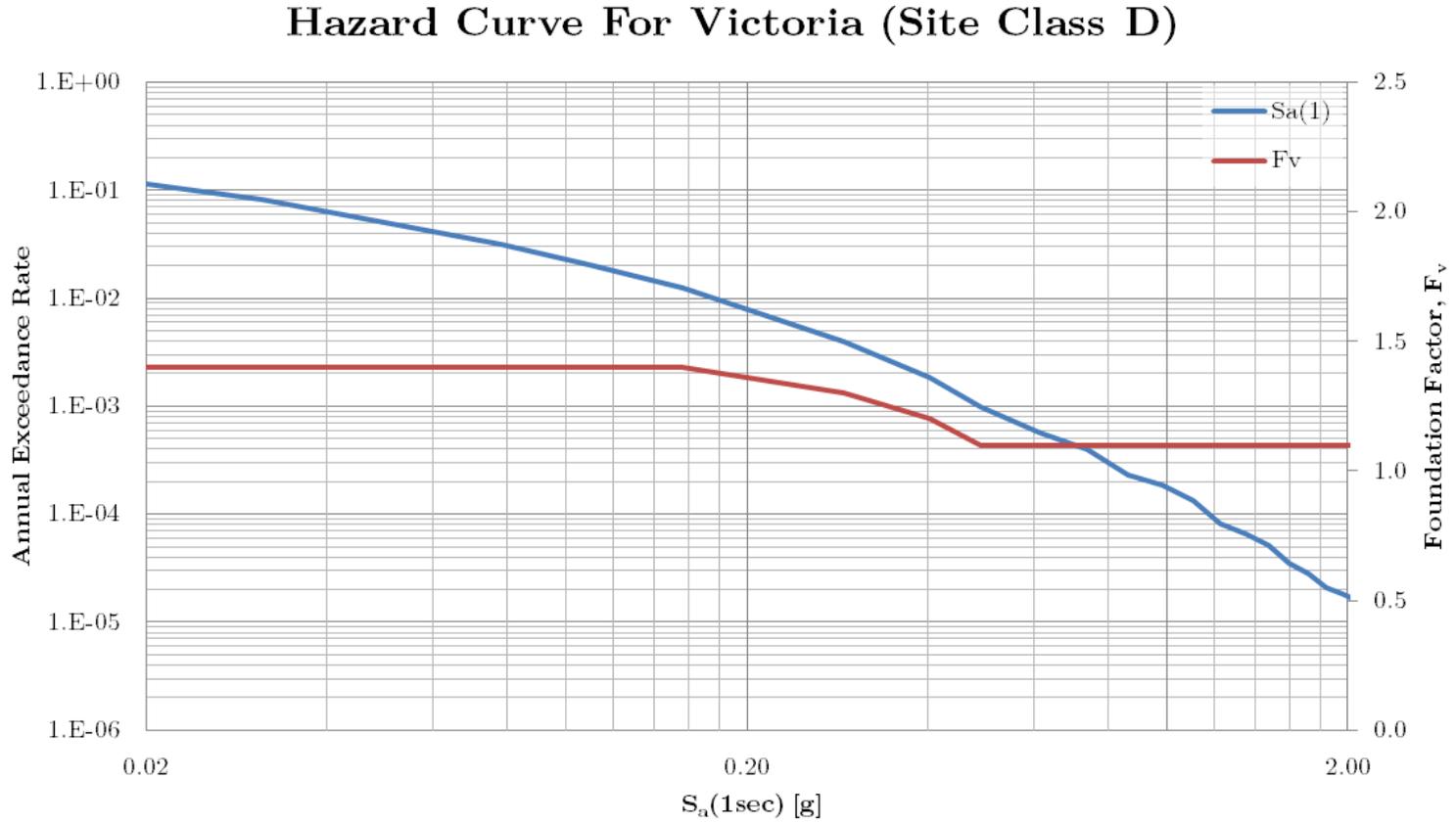


Figure D.3 – Seismic Hazard Data (Victoria, Site Class D)

D.4 Site Class E

The resulting hazard values for Site Class E are shown in Table D.4 and Figure D.4.

Table D.4 – Seismic Hazard Data (Victoria, Site Class E)

Annual Probability of Exceedance	$S_a(1)$ for RGC (Site Class C) [g]	F_v	$F_v * S_a(1)$
0.150400	0.01	2.1	0.02
0.082400	0.02	2.1	0.05
0.031510	0.06	2.1	0.12
0.020510	0.08	2.1	0.16
0.012370	0.11	2.1	0.23
0.003974	0.22	2.0	0.45
0.001822	0.34	1.9	0.64
0.000975	0.45	1.7	0.76
0.000571	0.56	1.7	0.95
0.000403	0.67	1.7	1.14
0.000234	0.78	1.7	1.33
0.000183	0.89	1.7	1.52
0.000132	1.01	1.7	1.71
0.000082	1.12	1.7	1.90
0.000066	1.23	1.7	2.09
0.000051	1.34	1.7	2.28
0.000035	1.45	1.7	2.47
0.000028	1.56	1.7	2.66
0.000021	1.68	1.7	2.85
0.000018	1.79	1.7	3.04
0.000016	1.90	1.7	3.23
0.000013	2.01	1.7	3.42
0.000011	2.12	1.7	3.61
0.000008	2.23	1.7	3.80

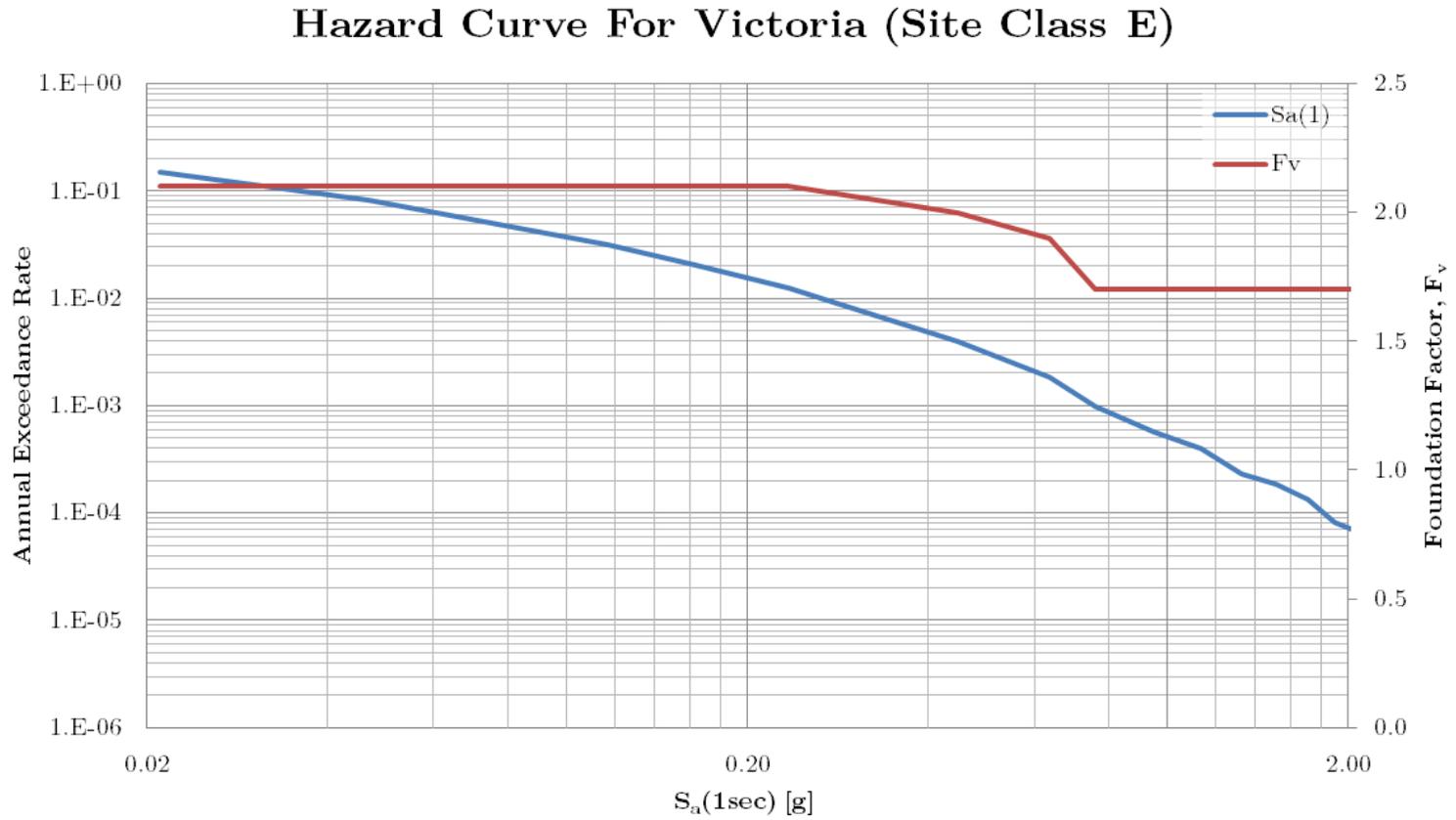


Figure D.4 – Seismic Hazard Data (Victoria, Site Class E)

Appendix E

Cost-Benefit Analysis Calculations

Cost-benefit analyses for various seismic rehabilitation measures were performed, as presented in Chapter 5, Section 5.9. Three upgrading measures were considered: parapet bracing, partial rehabilitation (tension ties at all floors), and full rehabilitation (consistent with local practices in Victoria, BC), each represented by separate motion-damage relationships, as presented in Section 5.6. The cost for each retrofitting measure was as discussed in Section 5.8. The analyses were in terms of annual expected cost (converted to present value), based on the seismic hazard curve for Victoria, BC. Because of the importance of site soils, the analyses were performed for Site Classes B through E, which represents a practical range of conditions. The three strengthening measures and four site classes yield a total of 12 cost-benefit results for the base case. Results for the sensitivity analysis were calculated similarly and are not provided here. The figures on the following pages provide the calculations for the 12 base case analyses. See section 5.9 for a summary of the results.

Cost-Benefit Analysis for Seismic Upgrades

Site Class= B																
RGC (Class 'C')			Adjusted		UNRETROFITTED						PARAPETS BRACED					
Sa(1)	Ann. Exceedance	Fv	Sa(1)	%/50 Yrs	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days
0.01	0.150400	0.6	0.01	99.9%	0.00%	0.0000%	0.00000	0.00000	0.00000	0.00000	0.00%	0.0000%	0.00000	0.00000	0.00000	0.00000
0.02	0.082400	0.6	0.01	98.4%	0.03%	0.0017%	0.00000	0.00000	0.00120	0.00012	0.01%	0.0006%	0.00000	0.00000	0.00001	0.00000
0.06	0.031510	0.6	0.03	79.3%	0.32%	0.0035%	0.00000	0.00000	0.12967	0.00278	0.19%	0.0021%	0.00000	0.00000	0.00676	0.00034
0.08	0.020510	0.6	0.05	64.1%	0.59%	0.0048%	0.00000	0.00000	0.34963	0.00527	0.40%	0.0032%	0.00000	0.00000	0.02862	0.00090
0.11	0.012370	0.6	0.07	46.1%	1.35%	0.0113%	0.00000	0.00000	1.77122	0.01753	1.01%	0.0085%	0.00000	0.00000	0.25655	0.00441
0.22	0.003974	0.7	0.16	18.0%	6.94%	0.0149%	0.00000	0.00000	6.34706	0.03610	5.96%	0.0128%	0.00000	0.00000	2.52200	0.01673
0.34	0.001822	0.7	0.23	8.7%	13.13%	0.0111%	0.00000	0.00000	5.58582	0.02990	11.56%	0.0098%	0.00000	0.00000	3.10824	0.01678
0.45	0.000975	0.8	0.36	4.8%	22.62%	0.0091%	0.00000	0.00000	4.64441	0.03429	20.02%	0.0081%	0.00000	0.00000	3.31309	0.01689
0.56	0.000571	0.8	0.45	2.8%	26.42%	0.0045%	0.00000	0.00000	2.21762	0.01643	23.36%	0.0039%	0.00000	0.00000	1.68364	0.01200
0.67	0.000403	0.8	0.54	2.0%	33.57%	0.0057%	0.00000	0.00000	2.65805	0.02850	29.58%	0.0050%	0.00000	0.00000	2.20127	0.01533
0.78	0.000234	0.8	0.63	1.2%	40.02%	0.0020%	0.00000	0.00000	0.89625	0.00973	35.20%	0.0018%	0.00000	0.00000	0.78488	0.00559
0.89	0.000183	0.8	0.71	0.9%	45.77%	0.0023%	0.00000	0.00000	0.96574	0.01800	40.26%	0.0020%	0.00000	0.00000	0.87800	0.00898
1.01	0.000132	0.8	0.80	0.7%	50.86%	0.0051%	0.00000	0.00000	1.01636	0.01896	44.81%	0.0023%	0.00000	0.00000	0.94820	0.00989
1.12	0.000082	0.8	0.89	0.4%	53.18%	0.0015%	0.00000	0.00000	0.31392	0.00588	46.92%	0.0007%	0.00000	0.00000	0.29577	0.00312
1.23	0.000066	0.8	0.98	0.3%	57.41%	0.0015%	0.00000	0.00000	0.32363	0.00612	50.84%	0.0015%	0.00000	0.00000	0.30965	0.00558
1.34	0.000051	0.8	1.07	0.3%	61.15%	0.0015%	0.00000	0.00000	0.33089	0.00959	54.39%	0.0015%	0.00000	0.00000	0.32014	0.00582
1.45	0.000035	0.8	1.16	0.2%	64.47%	0.0007%	0.00000	0.00000	0.16192	0.00470	57.63%	0.0007%	0.00000	0.00000	0.15794	0.00290
1.56	0.000028	0.8	1.25	0.1%	67.42%	0.0007%	0.00000	0.00000	0.16393	0.00478	60.59%	0.0007%	0.00000	0.00000	0.16087	0.00299
1.68	0.000021	0.8	1.34	0.1%	68.78%	0.0003%	0.00000	0.00000	0.05579	0.00163	61.98%	0.0003%	0.00000	0.00000	0.05488	0.00156
1.79	0.000018	0.8	1.43	0.1%	71.27%	0.0003%	0.00000	0.00000	0.05625	0.00165	64.58%	0.0003%	0.00000	0.00000	0.05555	0.00159
1.90	0.000016	0.8	1.52	0.1%	73.51%	0.0003%	0.00000	0.00000	0.05662	0.00167	66.97%	0.0003%	0.00000	0.00000	0.05607	0.00161
2.01	0.000013	0.8	1.61	0.1%	75.53%	0.0003%	0.00000	0.00000	0.05690	0.00168	69.17%	0.0003%	0.00000	0.00000	0.05648	0.00163
2.12	0.000011	0.8	1.70	0.1%	76.46%	0.0003%	0.00000	0.00000	0.05702	0.00169	70.20%	0.0003%	0.00000	0.00000	0.05665	0.00164
2.23	0.000008	0.8	1.79	0.0%	78.20%	0.0000%	0.00000	0.00000	0.00098	0.00003	72.16%	0.0000%	0.00000	0.00000	0.00098	0.00003
Building Data					Σ Ann. MDF= 0.08345% 0.00003 0.00003 0.25701						Σ Ann. MDF= 0.06673% 0.00001 0.00001 0.13632					
Cost (\$CAN) = \$260/SF					Ann. Cost= \$1,736 \$382 \$398 \$28 \$137						Ann. Cost= \$1,388 \$154 \$189 \$17 \$73					
Typical SF= 8000					50-Yr Cost= \$86,790 \$19,075 \$19,881 \$1,408 \$6,854						50-Yr Cost= \$69,399 \$7,683 \$9,431 \$863 \$3,635					
Repl. Value= \$2,080,000					PV= \$31,689 \$9,816 \$10,231 \$514 \$2,502						PV= \$25,339 \$3,954 \$4,853 \$315 \$1,327					
Full Upgrade Cost= \$33/SF											Savings= \$6,350 \$5,862 \$5,377 \$199 \$1,175					
Partial Upgrade Cost= \$10/SF											Total Savings= \$18,963					
Parapet Upgrade Cost= \$3/SF											Upgrade Cost= \$24,000					
Value of Life= \$9,100,000											B/C Ratio= 0.79					
Relocation Expense= \$2.90/SF																
Rental Cost= \$0.07/SF/day																
Occupancy Data					Economic Parameters											
Average Occupancy Density= .0036 Occ/SF					Time Horizon= 50 Years											
Average No. of Occupants= 28.8					Discount Rate for Capital= 5%											
Typical Streetfront Exposure= 30. lin. ft					Discount Rate for Life-Safety= 3%											
Average Pedestrian Density= .03 Ped./ft																
Average No. of Pedestrians= 0.9																
#of serious injuries/fatalities= 4																
%VSL assigned to injuries= 0.15																

Figure E.1 – Parapet Bracing (Site Class B)

Cost-Benefit Analysis for Seismic Upgrades

Site Class= B					UNRETROFITTED						PARTIAL RETROFIT					
Sa(1)	RGC (Class 'C') Ann. Exceedance	Fv	Adjusted Sa(1)	%/50 Yrs	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days
0.01	0.150400	0.6	0.01	99.9%	0.00%	0.0000%	0.00000	0.00000	0.00000	0.00000	0.00%	0.0000%	0.00000	0.00000	0.00000	0.00000
0.02	0.082400	0.6	0.01	98.4%	0.03%	0.0017%	0.00000	0.00000	0.00120	0.00012	0.01%	0.0005%	0.00000	0.00000	0.00000	0.00000
0.06	0.031510	0.6	0.03	79.3%	0.32%	0.0035%	0.00000	0.00000	0.12967	0.00278	0.14%	0.0015%	0.00000	0.00000	0.00431	0.00033
0.08	0.020510	0.6	0.05	64.1%	0.59%	0.0048%	0.00000	0.00000	0.34963	0.00527	0.28%	0.0023%	0.00000	0.00000	0.01903	0.00086
0.11	0.012370	0.6	0.07	46.1%	1.35%	0.0113%	0.00000	0.00000	1.77122	0.01753	0.73%	0.0061%	0.00000	0.00000	0.18100	0.00415
0.22	0.003974	0.7	0.16	18.0%	6.94%	0.0149%	0.00000	0.00000	6.34706	0.03610	4.58%	0.0099%	0.00000	0.00000	2.02442	0.01490
0.34	0.001822	0.7	0.23	8.7%	13.13%	0.0111%	0.00000	0.00000	5.58582	0.02990	9.28%	0.0079%	0.00000	0.00000	2.63838	0.01488
0.45	0.000975	0.8	0.36	4.8%	22.62%	0.0091%	0.00000	0.00000	4.64441	0.03429	16.85%	0.0068%	0.00000	0.00000	2.95694	0.01509
0.56	0.000571	0.8	0.45	2.8%	26.42%	0.0045%	0.00000	0.00000	2.21762	0.01643	19.95%	0.0034%	0.00000	0.00000	1.52477	0.01107
0.67	0.000403	0.8	0.54	2.0%	33.57%	0.0057%	0.00000	0.00000	2.65805	0.02850	25.85%	0.0044%	0.00000	0.00000	2.03911	0.01413
0.78	0.000234	0.8	0.63	1.2%	40.02%	0.0020%	0.00000	0.00000	0.89625	0.00973	31.26%	0.0016%	0.00000	0.00000	0.73916	0.00515
0.89	0.000183	0.8	0.71	0.9%	45.77%	0.0023%	0.00000	0.00000	0.96574	0.01800	36.17%	0.0018%	0.00000	0.00000	0.83717	0.00846
1.01	0.000132	0.8	0.80	0.7%	50.86%	0.0051%	0.00000	0.00000	1.01636	0.01896	40.61%	0.0021%	0.00000	0.00000	0.91272	0.00932
1.12	0.000082	0.8	0.89	0.4%	53.18%	0.0015%	0.00000	0.00000	0.31392	0.00588	42.68%	0.0007%	0.00000	0.00000	0.28582	0.00295
1.23	0.000066	0.8	0.98	0.3%	57.41%	0.0015%	0.00000	0.00000	0.32363	0.00612	46.52%	0.0007%	0.00000	0.00000	0.30120	0.00539
1.34	0.000051	0.8	1.07	0.3%	61.15%	0.0015%	0.00000	0.00000	0.33089	0.00959	50.02%	0.0015%	0.00000	0.00000	0.31300	0.00562
1.45	0.000035	0.8	1.16	0.2%	64.47%	0.0007%	0.00000	0.00000	0.16192	0.00470	53.22%	0.0007%	0.00000	0.00000	0.15505	0.00281
1.56	0.000028	0.8	1.25	0.1%	67.42%	0.0007%	0.00000	0.00000	0.16393	0.00478	56.16%	0.0007%	0.00000	0.00000	0.15844	0.00290
1.68	0.000021	0.8	1.34	0.1%	68.78%	0.0003%	0.00000	0.00000	0.05579	0.00163	57.54%	0.0003%	0.00000	0.00000	0.05413	0.00154
1.79	0.000018	0.8	1.43	0.1%	71.27%	0.0003%	0.00000	0.00000	0.05625	0.00165	60.14%	0.0003%	0.00000	0.00000	0.05492	0.00156
1.90	0.000016	0.8	1.52	0.1%	73.51%	0.0003%	0.00000	0.00000	0.05662	0.00167	62.55%	0.0003%	0.00000	0.00000	0.05554	0.00159
2.01	0.000013	0.8	1.61	0.1%	75.53%	0.0003%	0.00000	0.00000	0.05690	0.00168	64.78%	0.0003%	0.00000	0.00000	0.05603	0.00161
2.12	0.000011	0.8	1.70	0.1%	76.46%	0.0003%	0.00000	0.00000	0.05702	0.00169	65.84%	0.0003%	0.00000	0.00000	0.05624	0.00162
2.23	0.000008	0.8	1.79	0.0%	78.20%	0.0000%	0.00000	0.00000	0.00098	0.00003	67.84%	0.0000%	0.00000	0.00000	0.00097	0.00003
Building Data					Σ Ann. MDF=	0.08345%	0.00003	0.00003		0.25701	Σ Ann. MDF=	0.05392%	0.00001	0.00001		0.12595
Cost (\$CAN) = \$260/SF					Ann. Cost=	\$1,736	\$382	\$398	\$28	\$137	Ann. Cost=	\$1,121	\$98	\$155	\$15	\$67
Typical SF= 8000					50-Yr Cost=	\$86,790	\$19,075	\$19,881	\$1,408	\$6,854	50-Yr Cost=	\$56,074	\$4,917	\$7,764	\$768	\$3,359
Repl. Value= \$2,080,000					PV=	\$31,689	\$9,816	\$10,231	\$514	\$2,502	PV=	\$20,474	\$2,530	\$3,996	\$281	\$1,226
Full Upgrade Cost= \$33/SF											Savings=	\$11,215	\$7,286	\$6,235	\$234	\$1,276
Partial Upgrade Cost= \$10/SF											Total Savings=	\$26,245				
Parapet Upgrade Cost= \$3/SF											Upgrade Cost=	\$80,000				
Value of Life= \$9,100,000											B/C Ratio=	0.33				
Relocation Expense= \$2.90/SF																
Rental Cost= \$0.07/SF/day																

Occupancy Data
 Average Occupancy Density= .0036 Occ/SF
 Average No. of Occupants= 28.8
 Typical Streetfront Exposure= 30. lin. ft
 Average Pedestrian Density= .03 Ped./ft
 Average No. of Pedestrians= 0.9
 #of serious injuries/fatalities= 4
 %VSL assigned to injuries= 0.15

Economic Parameters
 Time Horizon= 50 Years
 Discount Rate for Capital= 5%
 Discount Rate for Life-Safety= 3%

Figure E.2 – Partial Retrofit (Site Class B)

Cost-Benefit Analysis for Seismic Upgrades

Site Class= B		UNRETROFITTED													RETROFITTED				
RGC (Class 'C')		Adjusted		UNRETROFITTED							RETROFITTED								
Sa(1)	Ann. Exceedance	Fv	Sa(1)	%/50 Yrs	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days			
0.01	0.150400	0.6	0.01	99.9%	0.00%	0.0000%	0.00000	0.00000	0.00000	0.00000	0.00%	0.0000%	0.00000	0.00000	0.00000	0.00000			
0.02	0.082400	0.6	0.01	98.4%	0.03%	0.0017%	0.00000	0.00000	0.00120	0.00012	0.01%	0.0003%	0.00000	0.00000	0.00011	0.00003			
0.06	0.031510	0.6	0.03	79.3%	0.32%	0.0035%	0.00000	0.00000	0.12967	0.00278	0.10%	0.0011%	0.00000	0.00000	0.01592	0.00072			
0.08	0.020510	0.6	0.05	64.1%	0.59%	0.0048%	0.00000	0.00000	0.34963	0.00527	0.20%	0.0016%	0.00000	0.00000	0.04789	0.00145			
0.11	0.012370	0.6	0.07	46.1%	1.35%	0.0113%	0.00000	0.00000	1.77122	0.01753	0.49%	0.0041%	0.00000	0.00000	0.28995	0.00521			
0.22	0.003974	0.7	0.16	18.0%	6.94%	0.0149%	0.00000	0.00000	6.34706	0.03610	2.99%	0.0064%	0.00000	0.00000	1.69857	0.01280			
0.34	0.001822	0.7	0.23	8.7%	13.13%	0.0111%	0.00000	0.00000	5.58582	0.02990	6.07%	0.0051%	0.00000	0.00000	1.93443	0.01168			
0.45	0.000975	0.8	0.36	4.8%	22.62%	0.0091%	0.00000	0.00000	4.64441	0.03429	11.30%	0.0046%	0.00000	0.00000	2.07886	0.01135			
0.56	0.000571	0.8	0.45	2.8%	26.42%	0.0045%	0.00000	0.00000	2.21762	0.01643	13.64%	0.0023%	0.00000	0.00000	1.07512	0.00902			
0.67	0.000403	0.8	0.54	2.0%	33.57%	0.0057%	0.00000	0.00000	2.65805	0.02850	18.52%	0.0031%	0.00000	0.00000	1.46608	0.01129			
0.78	0.000234	0.8	0.63	1.2%	40.02%	0.0020%	0.00000	0.00000	0.89625	0.00973	23.55%	0.0012%	0.00000	0.00000	0.54603	0.00639			
0.89	0.000183	0.8	0.71	0.9%	45.77%	0.0023%	0.00000	0.00000	0.96574	0.01800	28.53%	0.0014%	0.00000	0.00000	0.63647	0.00710			
1.01	0.000132	0.8	0.80	0.7%	50.86%	0.0051%	0.00000	0.00000	1.01636	0.01896	33.31%	0.0017%	0.00000	0.00000	0.71352	0.00777			
1.12	0.000082	0.8	0.89	0.4%	53.18%	0.0015%	0.00000	0.00000	0.31392	0.00588	35.59%	0.0005%	0.00000	0.00000	0.22642	0.00245			
1.23	0.000066	0.8	0.98	0.3%	57.41%	0.0015%	0.00000	0.00000	0.32363	0.00612	39.91%	0.0006%	0.00000	0.00000	0.24459	0.00478			
1.34	0.000051	0.8	1.07	0.3%	61.15%	0.0015%	0.00000	0.00000	0.33089	0.00959	43.85%	0.0007%	0.00000	0.00000	0.25993	0.00498			
1.45	0.000035	0.8	1.16	0.2%	64.47%	0.0007%	0.00000	0.00000	0.16192	0.00470	47.44%	0.0004%	0.00000	0.00000	0.13137	0.00248			
1.56	0.000028	0.8	1.25	0.1%	67.42%	0.0007%	0.00000	0.00000	0.16393	0.00478	50.67%	0.0007%	0.00000	0.00000	0.13667	0.00256			
1.68	0.000021	0.8	1.34	0.1%	68.78%	0.0003%	0.00000	0.00000	0.05579	0.00163	52.17%	0.0003%	0.00000	0.00000	0.04707	0.00141			
1.79	0.000018	0.8	1.43	0.1%	71.27%	0.0003%	0.00000	0.00000	0.05625	0.00165	54.94%	0.0003%	0.00000	0.00000	0.04848	0.00144			
1.90	0.000016	0.8	1.52	0.1%	73.51%	0.0003%	0.00000	0.00000	0.05662	0.00167	57.44%	0.0003%	0.00000	0.00000	0.04968	0.00146			
2.01	0.000013	0.8	1.61	0.1%	75.53%	0.0003%	0.00000	0.00000	0.05690	0.00168	59.71%	0.0003%	0.00000	0.00000	0.05071	0.00148			
2.12	0.000011	0.8	1.70	0.1%	76.46%	0.0003%	0.00000	0.00000	0.05702	0.00169	60.77%	0.0003%	0.00000	0.00000	0.05117	0.00149			
2.23	0.000008	0.8	1.79	0.0%	78.20%	0.0000%	0.00000	0.00000	0.00098	0.00003	62.74%	0.0000%	0.00000	0.00000	0.00089	0.00003			
Building Data					Σ Ann. MDF=	0.08345%	0.00003	0.00003		0.25701	Σ Ann. MDF=	0.03726%	0.00001	0.00001		0.10938			
Cost (\$CAN) = \$260/SF					Ann. Cost=	\$1,736	\$382	\$398	\$28	\$137	Ann. Cost=	\$775	\$89	\$118	\$12	\$58			
Typical SF= 8000					50-Yr Cost=	\$86,790	\$19,075	\$19,881	\$1,408	\$6,854	50-Yr Cost=	\$38,748	\$4,453	\$5,894	\$587	\$2,917			
Repl. Value= \$2,080,000					PV=	\$31,689	\$9,816	\$10,231	\$514	\$2,502	PV=	\$14,147	\$2,291	\$3,033	\$215	\$1,065			
Full Upgrade Cost= \$33/SF					Savings=		\$17,541				Savings=		\$17,541	\$7,525	\$7,198	\$300	\$1,437		
Partial Upgrade Cost= \$10/SF					Total Savings=		\$34,000				Total Savings=		\$34,000						
Parapet Upgrade Cost= \$3/SF					Upgrade Cost=		\$264,000				Upgrade Cost=		\$264,000						
Value of Life= \$9,100,000					B/C Ratio=		0.13				B/C Ratio=		0.13						
Relocation Expense= \$2.90/SF																			
Rental Cost= \$0.07/SF/day																			
Occupancy Data					Economic Parameters														
Average Occupancy Density= .0036 Occ/SF					Time Horizon= 50 Years														
Average No. of Occupants= 28.8					Discount Rate for Capital= 5%														
Typical Streetfront Exposure= 30. lin. ft					Discount Rate for Life-Safety= 3%														
Average Pedestrian Density= .03 Ped./ft																			
Average No. of Pedestrians= 0.9																			
#of serious injuries/fatalities= 4																			
%VSL assigned to injuries= 0.15																			

Figure E.3 – Full Retrofit (Site Class B)

Cost-Benefit Analysis for Seismic Upgrades

Site Class= C					UNRETROFITTED						PARAPETS BRACED					
Sa(1)	RGC (Class 'C') Ann. Exceedance	Fv	Adjusted Sa(1)	%/50 Yrs	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days
0.01	0.150400	1.0	0.01	99.9%	0.03%	0.0023%	0.00000	0.00000	0.00160	0.00017	0.01%	0.0008%	0.00000	0.00000	0.00001	0.00000
0.02	0.082400	1.0	0.02	98.4%	0.14%	0.0070%	0.00000	0.00000	0.07669	0.00284	0.07%	0.0035%	0.00000	0.00000	0.00199	0.00020
0.06	0.031510	1.0	0.06	79.3%	1.13%	0.0125%	0.00000	0.00000	1.68671	0.01812	0.83%	0.0092%	0.00000	0.00000	0.21753	0.00422
0.08	0.020510	1.0	0.08	64.1%	1.82%	0.0148%	0.00000	0.00000	2.93306	0.02540	1.41%	0.0115%	0.00000	0.00000	0.51737	0.00725
0.11	0.012370	1.0	0.11	46.1%	4.15%	0.0348%	0.00000	0.00000	11.71409	0.07528	3.45%	0.0290%	0.00000	0.00000	3.45577	0.02935
0.22	0.003974	1.0	0.22	18.0%	12.33%	0.0265%	0.00000	0.00001	13.20764	0.07074	10.84%	0.0233%	0.00000	0.00000	7.12471	0.03901
0.34	0.001822	1.0	0.34	8.7%	18.70%	0.0158%	0.00000	0.00001	8.14515	0.06116	16.55%	0.0140%	0.00000	0.00000	5.35377	0.02742
0.45	0.000975	1.0	0.45	4.8%	26.42%	0.0107%	0.00000	0.00001	5.30676	0.03932	23.36%	0.0094%	0.00000	0.00000	4.02895	0.02872
0.56	0.000571	1.0	0.56	2.8%	36.88%	0.0062%	0.00000	0.00000	2.83051	0.03046	32.47%	0.0055%	0.00000	0.00000	2.41752	0.01696
0.67	0.000403	1.0	0.67	2.0%	42.98%	0.0073%	0.00001	0.00001	3.10176	0.05801	37.80%	0.0064%	0.00000	0.00000	2.77263	0.02825
0.78	0.000234	1.0	0.78	1.2%	48.40%	0.0025%	0.00000	0.00000	0.99302	0.01849	42.59%	0.0022%	0.00000	0.00000	0.91560	0.00944
0.89	0.000183	1.0	0.89	0.9%	53.18%	0.0051%	0.00000	0.00000	1.03638	0.01940	46.92%	0.0024%	0.00000	0.00000	0.97647	0.01032
1.01	0.000132	1.0	1.01	0.7%	59.34%	0.0051%	0.00001	0.00000	1.08127	0.03135	52.66%	0.0051%	0.00000	0.00000	1.04080	0.01883
1.12	0.000082	1.0	1.12	0.4%	62.86%	0.0015%	0.00000	0.00000	0.33382	0.00969	56.05%	0.0015%	0.00000	0.00000	0.32439	0.00592
1.23	0.000066	1.0	1.23	0.3%	65.99%	0.0015%	0.00000	0.00000	0.33861	0.00985	59.15%	0.0015%	0.00000	0.00000	0.33135	0.00613
1.34	0.000051	1.0	1.34	0.3%	68.78%	0.0015%	0.00000	0.00000	0.34228	0.00999	61.98%	0.0015%	0.00000	0.00000	0.33668	0.00960
1.45	0.000035	1.0	1.45	0.2%	72.42%	0.0007%	0.00000	0.00000	0.16670	0.00489	65.80%	0.0007%	0.00000	0.00000	0.16487	0.00473
1.56	0.000028	1.0	1.56	0.1%	74.55%	0.0007%	0.00000	0.00000	0.16765	0.00494	68.09%	0.0007%	0.00000	0.00000	0.16623	0.00479
1.68	0.000021	1.0	1.68	0.1%	76.46%	0.0003%	0.00000	0.00000	0.05702	0.00169	70.20%	0.0003%	0.00000	0.00000	0.05665	0.00164
1.79	0.000018	1.0	1.79	0.1%	78.20%	0.0003%	0.00000	0.00000	0.05722	0.00170	72.16%	0.0003%	0.00000	0.00000	0.05693	0.00166
1.90	0.000016	1.0	1.90	0.1%	79.77%	0.0003%	0.00000	0.00000	0.05739	0.00171	73.97%	0.0003%	0.00000	0.00000	0.05716	0.00167
2.01	0.000013	1.0	2.01	0.1%	81.87%	0.0003%	0.00000	0.00000	0.05757	0.00172	76.43%	0.0003%	0.00000	0.00000	0.05742	0.00169
2.12	0.000011	1.0	2.12	0.1%	83.12%	0.0003%	0.00000	0.00000	0.05767	0.00173	77.93%	0.0003%	0.00000	0.00000	0.05755	0.00170
2.23	0.000008	1.0	2.23	0.0%	84.26%	0.0000%	0.00000	0.00000	0.00099	0.00003	79.32%	0.0000%	0.00000	0.00000	0.00099	0.00003
Building Data					Σ Ann. MDF=	0.15784%	0.00005	0.00005		0.49868	Σ Ann. MDF=	0.12950%	0.00002	0.00002		0.25953
Cost (SCAN) = \$260/SF					Ann. Cost=	\$3,283	\$687	\$697	\$54	\$266	Ann. Cost=	\$2,694	\$299	\$331	\$30	\$138
Typical SF= 8000					50-Yr Cost=	\$164,153	\$34,359	\$34,837	\$2,688	\$13,298	50-Yr Cost=	\$134,679	\$14,951	\$16,541	\$1,522	\$6,921
Repl. Value= \$2,080,000					PV=	\$59,935	\$17,681	\$17,927	\$981	\$4,855	PV=	\$49,174	\$7,694	\$8,512	\$556	\$2,527
Full Upgrade Cost= \$33/SF					Savings= \$10,761											
Partial Upgrade Cost= \$10/SF					Total Savings= \$32,918											
Parapet Upgrade Cost= \$3/SF					Upgrade Cost= \$24,000											
Value of Life= \$9,100,000					B/C Ratio= 1.37											
Relocation Expenses= \$2.90/SF																
Rental Cost= \$0.07/SF/day																
Occupancy Data																
Average Occupancy Density= .0036 Occ/SF																
Average No. of Occupants= 28.8																
Typical Streetfront Exposure= 30. lin. ft																
Average Pedestrian Density= .03 Ped./ft																
Average No. of Pedestrians= 0.9																
#of serious injuries/fatalities= 4																
%VSL assigned to injuries= 0.15																
Economic Parameters																
Time Horizon= 50 Years																
Discount Rate for Capital= 5%																
Discount Rate for Life-Safety= 3%																

Figure E.4 – Parapet Bracing (Site Class C)

Cost-Benefit Analysis for Seismic Upgrades

Site Class= C																	
RGC (Class 'C')					UNRETROFITTED						PARTIAL RETROFIT						
Sa(1)	Ann. Exceedance	Fv	Adjusted Sa(1)	%/50 Yrs	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	
0.01	0.150400	1.0	0.01	99.9%	0.03%	0.0023%	0.00000	0.00000	0.00160	0.00017	0.01%	0.0007%	0.00000	0.00000	0.00001	0.00000	
0.02	0.082400	1.0	0.02	98.4%	0.14%	0.0070%	0.00000	0.00000	0.07669	0.00284	0.05%	0.0027%	0.00000	0.00000	0.00119	0.00020	
0.06	0.031510	1.0	0.06	79.3%	1.13%	0.0125%	0.00000	0.00000	1.68671	0.01812	0.60%	0.0066%	0.00000	0.00000	0.15154	0.00399	
0.08	0.020510	1.0	0.08	64.1%	1.82%	0.0148%	0.00000	0.00000	2.93306	0.02540	1.03%	0.0083%	0.00000	0.00000	0.37325	0.00675	
0.11	0.012370	1.0	0.11	46.1%	4.15%	0.0348%	0.00000	0.00000	11.71409	0.07528	2.58%	0.0217%	0.00000	0.00000	2.65830	0.02651	
0.22	0.003974	1.0	0.22	18.0%	12.33%	0.0265%	0.00000	0.00001	13.20764	0.07074	8.66%	0.0186%	0.00000	0.00000	6.01381	0.03459	
0.34	0.001822	1.0	0.34	8.7%	18.70%	0.0158%	0.00000	0.00001	8.14515	0.06116	13.69%	0.0116%	0.00000	0.00000	4.69423	0.02442	
0.45	0.000975	1.0	0.45	4.8%	26.42%	0.0107%	0.00000	0.00001	5.30676	0.03932	19.95%	0.0081%	0.00000	0.00000	3.64877	0.02650	
0.56	0.000571	1.0	0.56	2.8%	36.88%	0.0062%	0.00000	0.00000	2.83051	0.03046	28.62%	0.0048%	0.00000	0.00000	2.25936	0.01563	
0.67	0.000403	1.0	0.67	2.0%	42.98%	0.0073%	0.00001	0.00001	3.10176	0.05801	33.77%	0.0057%	0.00000	0.00000	2.62845	0.02663	
0.78	0.000234	1.0	0.78	1.2%	48.40%	0.0025%	0.00000	0.00000	0.99302	0.01849	38.44%	0.0020%	0.00000	0.00000	0.87744	0.00890	
0.89	0.000183	1.0	0.89	0.9%	53.18%	0.0051%	0.00000	0.00000	1.03638	0.01940	42.68%	0.0022%	0.00000	0.00000	0.94362	0.00972	
1.01	0.000132	1.0	1.01	0.7%	59.34%	0.0051%	0.00001	0.00000	1.08127	0.03135	48.31%	0.0025%	0.00000	0.00000	1.01513	0.01819	
1.12	0.000082	1.0	1.12	0.4%	62.86%	0.0015%	0.00000	0.00000	0.33382	0.00969	51.65%	0.0015%	0.00000	0.00000	0.31784	0.00573	
1.23	0.000066	1.0	1.23	0.3%	65.99%	0.0015%	0.00000	0.00000	0.33861	0.00985	54.72%	0.0015%	0.00000	0.00000	0.32585	0.00593	
1.34	0.000051	1.0	1.34	0.3%	68.78%	0.0015%	0.00000	0.00000	0.34228	0.00999	57.54%	0.0015%	0.00000	0.00000	0.33207	0.00943	
1.45	0.000035	1.0	1.45	0.2%	72.42%	0.0007%	0.00000	0.00000	0.16670	0.00489	61.37%	0.0007%	0.00000	0.00000	0.16316	0.00465	
1.56	0.000028	1.0	1.56	0.1%	74.55%	0.0007%	0.00000	0.00000	0.16765	0.00494	63.68%	0.0007%	0.00000	0.00000	0.16479	0.00472	
1.68	0.000021	1.0	1.68	0.1%	76.46%	0.0003%	0.00000	0.00000	0.05702	0.00169	65.84%	0.0003%	0.00000	0.00000	0.05624	0.00162	
1.79	0.000018	1.0	1.79	0.1%	78.20%	0.0003%	0.00000	0.00000	0.05722	0.00170	67.84%	0.0003%	0.00000	0.00000	0.05659	0.00163	
1.90	0.000016	1.0	1.90	0.1%	79.77%	0.0003%	0.00000	0.00000	0.05739	0.00171	69.70%	0.0003%	0.00000	0.00000	0.05687	0.00165	
2.01	0.000013	1.0	2.01	0.1%	81.87%	0.0003%	0.00000	0.00000	0.05757	0.00172	72.27%	0.0003%	0.00000	0.00000	0.05719	0.00167	
2.12	0.000011	1.0	2.12	0.1%	83.12%	0.0003%	0.00000	0.00000	0.05767	0.00173	73.84%	0.0003%	0.00000	0.00000	0.05735	0.00168	
2.23	0.000008	1.0	2.23	0.0%	84.26%	0.0000%	0.00000	0.00000	0.00099	0.00003	75.31%	0.0000%	0.00000	0.00000	0.00099	0.00003	
Building Data					Σ Ann. MDF=	0.15784%	0.00005	0.00005		0.49868	Σ Ann. MDF=	0.10278%	0.00001	0.00002		0.24076	
Cost (\$CAN) = \$260/SF					Ann. Cost=	\$3,283	\$687	\$697	\$54	\$266	Ann. Cost=	\$2,138	\$198	\$274	\$27	\$128	
Typical SF= 8000					50-Yr Cost=	\$164,153	\$34,359	\$34,837	\$2,688	\$13,298	50-Yr Cost=	\$106,894	\$9,886	\$13,715	\$1,343	\$6,420	
Repl. Value= \$2,080,000					PV=	\$59,935	\$17,681	\$17,927	\$981	\$4,855	PV=	\$39,029	\$5,087	\$7,057	\$490	\$2,344	
Full Upgrade Cost= \$33/SF											Savings=	\$20,906	\$12,594	\$10,869	\$491	\$2,511	
Partial Upgrade Cost= \$10/SF											Total Savings=	\$47,372					
Parapet Upgrade Cost= \$3/SF											Upgrade Cost=	\$80,000					
Value of Life= \$9,100,000											B/C Ratio=	0.59					
Relocation Expense= \$2.90/SF																	
Rental Cost= \$0.07/SF/day																	
Occupancy Data																	
Average Occupancy Density= .0036 Occ/SF																	
Average No. of Occupants= 28.8																	
Typical Streetfront Exposure= 30. lin. ft																	
Average Pedestrian Density= .03 Ped./ft																	
Average No. of Pedestrians= 0.9																	
#of serious injuries/fatalities= 4																	
%VSL assigned to injuries= 0.15																	
					Economic Parameters												
					Time Horizon= 50 Years												
					Discount Rate for Capital= 5%												
					Discount Rate for Life-Safety= 3%												

Figure E.5 – Partial Retrofit (Site Class C)

Cost-Benefit Analysis for Seismic Upgrades

Site Class= C																	
RGC (Class 'C')					UNRETROFITTED						RETROFITTED						
Sa(1)	Ann. Exceedance	Fv	Adjusted Sa(1)	%/50 Yrs	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	
0.01	0.150400	1.0	0.01	99.9%	0.03%	0.0023%	0.00000	0.00000	0.00160	0.00017	0.01%	0.0004%	0.00000	0.00000	0.00015	0.00004	
0.02	0.082400	1.0	0.02	98.4%	0.14%	0.0070%	0.00000	0.00000	0.07669	0.00284	0.04%	0.0019%	0.00000	0.00000	0.00828	0.00068	
0.06	0.031510	1.0	0.06	79.3%	1.13%	0.0125%	0.00000	0.00000	1.68671	0.01812	0.41%	0.0045%	0.00000	0.00000	0.26515	0.00530	
0.08	0.020510	1.0	0.08	64.1%	1.82%	0.0148%	0.00000	0.00000	2.93306	0.02540	0.68%	0.0056%	0.00000	0.00000	0.51740	0.00778	
0.11	0.012370	1.0	0.11	46.1%	4.15%	0.0348%	0.00000	0.00000	11.71409	0.07528	1.69%	0.0142%	0.00000	0.00000	2.62356	0.02508	
0.22	0.003974	1.0	0.22	18.0%	12.33%	0.0265%	0.00000	0.00001	13.20764	0.07074	5.66%	0.0122%	0.00000	0.00000	4.45119	0.02734	
0.34	0.001822	1.0	0.34	8.7%	18.70%	0.0158%	0.00000	0.00001	8.14515	0.06116	9.05%	0.0077%	0.00000	0.00000	3.31977	0.01856	
0.45	0.000975	1.0	0.45	4.8%	26.42%	0.0107%	0.00000	0.00001	5.30676	0.03932	13.64%	0.0055%	0.00000	0.00000	2.57278	0.02160	
0.56	0.000571	1.0	0.56	2.8%	36.88%	0.0062%	0.00000	0.00000	2.83051	0.03046	21.03%	0.0035%	0.00000	0.00000	1.64581	0.01243	
0.67	0.000403	1.0	0.67	2.0%	42.98%	0.0073%	0.00001	0.00001	3.10176	0.05801	26.05%	0.0044%	0.00000	0.00000	1.96979	0.02242	
0.78	0.000234	1.0	0.78	1.2%	48.40%	0.0025%	0.00000	0.00000	0.99302	0.01849	30.95%	0.0016%	0.00000	0.00000	0.67658	0.00744	
0.89	0.000183	1.0	0.89	0.9%	53.18%	0.0051%	0.00000	0.00000	1.03638	0.01940	35.59%	0.0018%	0.00000	0.00000	0.74751	0.00809	
1.01	0.000132	1.0	1.01	0.7%	59.34%	0.0051%	0.00001	0.00000	1.08127	0.03135	41.93%	0.0021%	0.00000	0.00000	0.83388	0.01611	
1.12	0.000082	1.0	1.12	0.4%	62.86%	0.0015%	0.00000	0.00000	0.33382	0.00969	45.69%	0.0007%	0.00000	0.00000	0.26670	0.00507	
1.23	0.000066	1.0	1.23	0.3%	65.99%	0.0015%	0.00000	0.00000	0.33861	0.00985	49.09%	0.0008%	0.00000	0.00000	0.27864	0.00524	
1.34	0.000051	1.0	1.34	0.3%	68.78%	0.0015%	0.00000	0.00000	0.34228	0.00999	52.17%	0.0015%	0.00000	0.00000	0.28878	0.00866	
1.45	0.000035	1.0	1.45	0.2%	72.42%	0.0007%	0.00000	0.00000	0.16670	0.00489	56.22%	0.0007%	0.00000	0.00000	0.14501	0.00428	
1.56	0.000028	1.0	1.56	0.1%	74.55%	0.0007%	0.00000	0.00000	0.16765	0.00494	58.61%	0.0007%	0.00000	0.00000	0.14829	0.00435	
1.68	0.000021	1.0	1.68	0.1%	76.46%	0.0003%	0.00000	0.00000	0.05702	0.00169	60.77%	0.0003%	0.00000	0.00000	0.05117	0.00149	
1.79	0.000018	1.0	1.79	0.1%	78.20%	0.0003%	0.00000	0.00000	0.05722	0.00170	62.74%	0.0003%	0.00000	0.00000	0.05199	0.00151	
1.90	0.000016	1.0	1.90	0.1%	79.77%	0.0003%	0.00000	0.00000	0.05739	0.00171	64.54%	0.0003%	0.00000	0.00000	0.05270	0.00153	
2.01	0.000013	1.0	2.01	0.1%	81.87%	0.0003%	0.00000	0.00000	0.05757	0.00172	66.97%	0.0003%	0.00000	0.00000	0.05359	0.00156	
2.12	0.000011	1.0	2.12	0.1%	83.12%	0.0003%	0.00000	0.00000	0.05767	0.00173	68.44%	0.0003%	0.00000	0.00000	0.05409	0.00157	
2.23	0.000008	1.0	2.23	0.0%	84.26%	0.0000%	0.00000	0.00000	0.00099	0.00003	69.81%	0.0000%	0.00000	0.00000	0.00094	0.00003	
Building Data					Σ Ann. MDF=	0.15784%	0.00005	0.00005	0.49868		Σ Ann. MDF=	0.07108%	0.00001	0.00001	0.20814		
Cost (\$CAN) = \$260/SF					Ann. Cost=	\$3,283	\$687	\$697	\$266	\$54	Ann. Cost=	\$1,479	\$172	\$211	\$21	\$111	
Typical SF= 8000					50-Yr Cost=	\$164,153	\$34,359	\$34,837	\$2,688	\$13,298	50-Yr Cost=	\$73,926	\$8,577	\$10,550	\$1,051	\$5,550	
Repl. Value= \$2,080,000					PV=	\$59,935	\$17,681	\$17,927	\$981	\$4,855	PV=	\$26,992	\$4,414	\$5,429	\$384	\$2,027	
Full Upgrade Cost= \$33/SF											Savings=	\$32,944	\$13,268	\$12,498	\$597	\$2,829	
Partial Upgrade Cost= \$10/SF											Total Savings=	\$62,135					
Parapet Upgrade Cost= \$3/SF											Upgrade Cost=	\$264,000					
Value of Life= \$9,100,000											B/C Ratio=	0.24					
Relocation Expense= \$2.90/SF																	
Rental Cost= \$0.07/SF/day																	

Occupancy Data
 Average Occupancy Density= 0.036 Occ/SF
 Average No. of Occupants= 28.8
 Typical Streetfront Exposure= 30. lin. ft
 Average Pedestrian Density= 0.3 Ped./ft
 Average No. of Pedestrians= 0.9
 #of serious injuries/fatalities= 4
 %VSL assigned to injuries= 0.15

Economic Parameters
 Time Horizon= 50 Years
 Discount Rate for Capital= 5%
 Discount Rate for Life-Safety= 3%

Figure E.6 – Full Retrofit (Site Class C)

Cost-Benefit Analysis for Seismic Upgrades

Site Class= D																
Sa(1)	RGC (Class 'C')		Adjusted		UNRETROFITTED						PARAPETS BRACED					
	Ann. Exceedance	Fv	Sa(1)	%/50 Yrs	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days
0.01	0.150400	1.4	0.01	99.9%	0.03%	0.0023%	0.00000	0.00000	0.00160	0.00017	0.01%	0.0008%	0.00000	0.00000	0.00001	0.00000
0.02	0.082400	1.4	0.03	98.4%	0.32%	0.0164%	0.00000	0.00000	0.59988	0.01284	0.19%	0.0099%	0.00000	0.00000	0.03128	0.00156
0.06	0.031510	1.4	0.08	79.3%	1.82%	0.0200%	0.00000	0.00000	3.96359	0.03432	1.41%	0.0155%	0.00000	0.00000	0.69915	0.00980
0.08	0.020510	1.4	0.11	64.1%	3.51%	0.0286%	0.00000	0.00000	8.76457	0.05927	2.88%	0.0235%	0.00000	0.00000	2.33832	0.02178
0.11	0.012370	1.4	0.16	46.1%	6.94%	0.0583%	0.00000	0.00001	24.76296	0.14085	5.96%	0.0500%	0.00000	0.00000	9.83955	0.06527
0.22	0.003974	1.3	0.29	18.0%	17.91%	0.0385%	0.00001	0.00001	19.82204	0.14990	15.85%	0.0341%	0.00000	0.00000	12.77804	0.06568
0.34	0.001822	1.2	0.40	8.7%	26.42%	0.0224%	0.00001	0.00001	11.13277	0.08248	23.36%	0.0198%	0.00000	0.00000	8.45211	0.06025
0.45	0.000975	1.1	0.49	4.8%	30.08%	0.0121%	0.00001	0.00001	5.87552	0.06328	26.55%	0.0107%	0.00000	0.00000	4.68245	0.03273
0.56	0.000571	1.1	0.61	2.8%	40.02%	0.0068%	0.00001	0.00001	2.97709	0.03232	35.20%	0.0059%	0.00000	0.00000	2.60717	0.01856
0.67	0.000403	1.1	0.74	2.0%	45.77%	0.0077%	0.00001	0.00001	3.20794	0.05978	40.26%	0.0068%	0.00000	0.00000	2.91650	0.02983
0.78	0.000234	1.1	0.86	1.2%	53.18%	0.0051%	0.00000	0.00000	1.03638	0.01940	46.92%	0.0024%	0.00000	0.00000	0.97647	0.01032
0.89	0.000183	1.1	0.98	0.9%	57.41%	0.0051%	0.00000	0.00000	1.06844	0.02019	50.84%	0.0051%	0.00000	0.00000	1.02229	0.01844
1.01	0.000132	1.1	1.11	0.7%	62.86%	0.0051%	0.00001	0.00000	1.10208	0.03198	56.05%	0.0051%	0.00000	0.00000	1.07094	0.01956
1.12	0.000082	1.1	1.23	0.4%	65.99%	0.0015%	0.00000	0.00000	0.33861	0.00985	59.15%	0.0015%	0.00000	0.00000	0.33135	0.00613
1.23	0.000066	1.1	1.35	0.3%	70.06%	0.0015%	0.00000	0.00000	0.34379	0.01005	63.30%	0.0015%	0.00000	0.00000	0.33887	0.00968
1.34	0.000051	1.1	1.47	0.3%	72.42%	0.0015%	0.00000	0.00000	0.34630	0.01016	65.80%	0.0015%	0.00000	0.00000	0.34250	0.00982
1.45	0.000035	1.1	1.60	0.2%	74.55%	0.0007%	0.00000	0.00000	0.16765	0.00494	68.09%	0.0007%	0.00000	0.00000	0.16623	0.00479
1.56	0.000028	1.1	1.72	0.1%	77.35%	0.0007%	0.00000	0.00000	0.16871	0.00500	71.20%	0.0007%	0.00000	0.00000	0.16774	0.00487
1.68	0.000021	1.1	1.84	0.1%	79.00%	0.0003%	0.00000	0.00000	0.05731	0.00170	73.08%	0.0003%	0.00000	0.00000	0.05705	0.00166
1.79	0.000018	1.1	1.97	0.1%	81.20%	0.0003%	0.00000	0.00000	0.05752	0.00172	75.64%	0.0003%	0.00000	0.00000	0.05734	0.00168
1.90	0.000016	1.1	2.09	0.1%	82.51%	0.0003%	0.00000	0.00000	0.05762	0.00173	77.19%	0.0003%	0.00000	0.00000	0.05749	0.00170
2.01	0.000013	1.1	2.21	0.1%	84.26%	0.0003%	0.00000	0.00000	0.05775	0.00174	79.32%	0.0003%	0.00000	0.00000	0.05766	0.00171
2.12	0.000011	1.1	2.33	0.1%	85.31%	0.0003%	0.00000	0.00000	0.05781	0.00175	80.61%	0.0003%	0.00000	0.00000	0.05774	0.00172
2.23	0.000008	1.1	2.46	0.0%	86.73%	0.0000%	0.00000	0.00000	0.00099	0.00003	82.38%	0.0000%	0.00000	0.00000	0.00099	0.00003
Building Data					Σ Ann. MDF=	0.23564%	0.00007	0.00007	0.75546		Σ Ann. MDF=	0.19694%	0.00003	0.00003	0.39757	
Cost (\$CAN) = \$260/SF					Ann. Cost=	\$4,901	\$994	\$1,074	\$86	\$403	Ann. Cost=	\$4,096	\$418	\$490	\$49	\$212
Typical SF= 8000					50-Yr Cost=	\$245,069	\$49,697	\$53,719	\$4,298	\$20,146	50-Yr Cost=	\$204,818	\$20,900	\$24,521	\$2,452	\$10,602
Repl. Value= \$2,080,000					PV=	\$89,479	\$25,574	\$27,644	\$1,569	\$7,356	PV=	\$74,783	\$10,755	\$12,618	\$895	\$3,871
Full Upgrade Cost= \$33/SF					Savings= \$14,696											
Partial Upgrade Cost= \$10/SF					\$14,819											
Parapet Upgrade Cost= \$3/SF					\$15,025											
Value of Life= \$9,100,000					\$674											
Relocation Expense= \$2.90/SF					\$3,485											
Rental Cost= \$0.07/SF/day					Total Savings= \$48,699											
					Upgrade Cost= \$24,000											
					B/C Ratio= 2.03											

Occupancy Data
 Average Occupancy Density= .0036 Occ/SF
 Average No. of Occupants= 28.8
 Typical Streetfront Exposure= 30. lin. ft
 Average Pedestrian Density= .03 Ped./ft
 Average No. of Pedestrians= 0.9
 #of serious injuries/fatalities= 4
 %VSL assigned to injuries= 0.15

Economic Parameters
 Time Horizon= 50 Years
 Discount Rate for Capital= 5%
 Discount Rate for Life-Safety= 3%

Figure E.7 – Parapet Bracing (Site Class D)

Cost-Benefit Analysis for Seismic Upgrades

Site Class= D																	
Sa(1)	RGC (Class 'C')		Adjusted			UNRETROFITTED						PARTIAL RETROFIT					
	Ann. Exceedance	Fv	Sa(1)	%/50 Yrs	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	
0.01	0.150400	1.4	0.01	99.9%	0.03%	0.0023%	0.00000	0.00000	0.00160	0.00017	0.01%	0.0007%	0.00000	0.00000	0.00001	0.00000	
0.02	0.082400	1.4	0.03	98.4%	0.32%	0.0164%	0.00000	0.00000	0.59988	0.01284	0.14%	0.0072%	0.00000	0.00000	0.01993	0.00153	
0.06	0.031510	1.4	0.08	79.3%	1.82%	0.0200%	0.00000	0.00000	3.96359	0.03432	1.03%	0.0113%	0.00000	0.00000	0.50440	0.00912	
0.08	0.020510	1.4	0.11	64.1%	3.51%	0.0286%	0.00000	0.00000	8.76457	0.05927	2.15%	0.0175%	0.00000	0.00000	1.77487	0.01978	
0.11	0.012370	1.4	0.16	46.1%	6.94%	0.0583%	0.00000	0.00001	24.76296	0.14085	4.58%	0.0385%	0.00000	0.00000	7.89826	0.05815	
0.22	0.003974	1.3	0.29	18.0%	17.91%	0.0385%	0.00001	0.00001	19.82204	0.14990	13.06%	0.0281%	0.00000	0.00000	11.15900	0.05844	
0.34	0.001822	1.2	0.40	8.7%	26.42%	0.0224%	0.00001	0.00001	11.13277	0.08248	19.95%	0.0169%	0.00000	0.00000	7.65456	0.05558	
0.45	0.000975	1.1	0.49	4.8%	30.08%	0.0121%	0.00001	0.00001	5.87552	0.06328	22.96%	0.0093%	0.00000	0.00000	4.29279	0.03017	
0.56	0.000571	1.1	0.61	2.8%	40.02%	0.0068%	0.00001	0.00001	2.97709	0.03232	31.26%	0.0053%	0.00000	0.00000	2.45529	0.01711	
0.67	0.000403	1.1	0.74	2.0%	45.77%	0.0077%	0.00001	0.00001	3.20794	0.05978	36.17%	0.0061%	0.00000	0.00000	2.78085	0.02811	
0.78	0.000234	1.1	0.86	1.2%	53.18%	0.0051%	0.00000	0.00000	1.03638	0.01940	42.68%	0.0022%	0.00000	0.00000	0.94362	0.00972	
0.89	0.000183	1.1	0.98	0.9%	57.41%	0.0051%	0.00000	0.00000	1.06844	0.02019	46.52%	0.0024%	0.00000	0.00000	0.99437	0.01781	
1.01	0.000132	1.1	1.11	0.7%	62.86%	0.0051%	0.00001	0.00000	1.10208	0.03198	51.65%	0.0051%	0.00000	0.00000	1.04933	0.01891	
1.12	0.000082	1.1	1.23	0.4%	65.99%	0.0015%	0.00000	0.00000	0.33861	0.00985	54.72%	0.0015%	0.00000	0.00000	0.32585	0.00593	
1.23	0.000066	1.1	1.35	0.3%	70.06%	0.0015%	0.00000	0.00000	0.34379	0.01005	58.87%	0.0015%	0.00000	0.00000	0.33465	0.00951	
1.34	0.000051	1.1	1.47	0.3%	72.42%	0.0015%	0.00000	0.00000	0.34630	0.01016	61.37%	0.0015%	0.00000	0.00000	0.33895	0.00966	
1.45	0.000035	1.1	1.60	0.2%	74.55%	0.0007%	0.00000	0.00000	0.16765	0.00494	63.68%	0.0007%	0.00000	0.00000	0.16479	0.00472	
1.56	0.000028	1.1	1.72	0.1%	77.35%	0.0007%	0.00000	0.00000	0.16871	0.00500	66.85%	0.0007%	0.00000	0.00000	0.16663	0.00480	
1.68	0.000021	1.1	1.84	0.1%	79.00%	0.0003%	0.00000	0.00000	0.05731	0.00170	68.79%	0.0003%	0.00000	0.00000	0.05674	0.00164	
1.79	0.000018	1.1	1.97	0.1%	81.20%	0.0003%	0.00000	0.00000	0.05752	0.00172	71.44%	0.0003%	0.00000	0.00000	0.05709	0.00166	
1.90	0.000016	1.1	2.09	0.1%	82.51%	0.0003%	0.00000	0.00000	0.05762	0.00173	73.07%	0.0003%	0.00000	0.00000	0.05728	0.00168	
2.01	0.000013	1.1	2.21	0.1%	84.26%	0.0003%	0.00000	0.00000	0.05775	0.00174	75.31%	0.0003%	0.00000	0.00000	0.05749	0.00169	
2.12	0.000011	1.1	2.33	0.1%	85.31%	0.0003%	0.00000	0.00000	0.05781	0.00175	76.69%	0.0003%	0.00000	0.00000	0.05760	0.00170	
2.23	0.000008	1.1	2.46	0.0%	86.73%	0.0000%	0.00000	0.00000	0.00099	0.00003	78.60%	0.0000%	0.00000	0.00000	0.00099	0.00003	
Building Data					Σ Ann. MDF=	0.23564%	0.00007	0.00007		0.75546	Σ Ann. MDF=	0.15771%	0.00002	0.00003		0.36746	
Cost (SCAN) = \$260/SF					Ann. Cost=	\$4,901	\$994	\$1,074	\$86	\$403	Ann. Cost=	\$3,280	\$276	\$404	\$43	\$196	
Typical SF= 8000					50-Yr Cost=	\$245,069	\$49,697	\$53,719	\$4,298	\$20,146	50-Yr Cost=	\$164,021	\$13,820	\$20,196	\$2,157	\$9,799	
Repl. Value= \$2,080,000					PV=	\$89,479	\$25,574	\$27,644	\$1,569	\$7,356	PV=	\$59,887	\$7,112	\$10,393	\$788	\$3,578	
Full Upgrade Cost= \$33/SF																	
Partial Upgrade Cost= \$10/SF																	
Parapet Upgrade Cost= \$3/SF																	
Value of Life= \$9,100,000																	
Relocation Expense= \$2.90/SF																	
Rental Cost= \$0.07/SF/day																	
Occupancy Data																	
Average Occupancy Density= .0036 Occ/SF																	
Average No. of Occupants= 28.8																	
Typical Streetfront Exposure= 30. lin. ft																	
Average Pedestrian Density= .03 Ped./ft																	
Average No. of Pedestrians= 0.9																	
#of serious injuries/fatalities= 4																	
%VSL assigned to injuries= 0.15																	
Economic Parameters																	
Time Horizon= 50 Years																	
Discount Rate for Capital= 5%																	
Discount Rate for Life-Safety= 3%																	
											Savings=	\$29,592	\$18,462	\$17,251	\$782	\$3,778	
											Total Savings=	\$69,865					
											Upgrade Cost=	\$80,000					
											B/C Ratio=	0.87					

Figure E.8 – Partial Retrofit (Site Class D)

Cost-Benefit Analysis for Seismic Upgrades

Site Class= D																
Sa(1)	RGC (Class 'C')		Adjusted Sa(1)	% /50 Yrs	UNRETROFITTED						RETROFITTED					
	Ann. Exceedance	Fv			MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days
0.01	0.150400	1.4	0.01	99.9%	0.03%	0.0023%	0.00000	0.00000	0.00160	0.00017	0.01%	0.0004%	0.00000	0.00000	0.00015	0.00004
0.02	0.082400	1.4	0.03	98.4%	0.32%	0.0164%	0.00000	0.00000	0.59988	0.01284	0.10%	0.0051%	0.00000	0.00000	0.07366	0.00332
0.06	0.031510	1.4	0.08	79.3%	1.82%	0.0200%	0.00000	0.00000	3.96359	0.03432	0.68%	0.0075%	0.00000	0.00000	0.69918	0.01051
0.08	0.020510	1.4	0.11	64.1%	3.51%	0.0286%	0.00000	0.00000	8.76457	0.05927	1.40%	0.0114%	0.00000	0.00000	1.86241	0.01939
0.11	0.012370	1.4	0.16	46.1%	6.94%	0.0583%	0.00000	0.00001	24.76296	0.14085	2.99%	0.0251%	0.00000	0.00000	6.62694	0.04994
0.22	0.003974	1.3	0.29	18.0%	17.91%	0.0385%	0.00001	0.00001	19.82204	0.14990	8.61%	0.0185%	0.00000	0.00000	7.91318	0.04457
0.34	0.001822	1.2	0.40	8.7%	26.42%	0.0224%	0.00001	0.00001	11.13277	0.08248	13.64%	0.0116%	0.00000	0.00000	5.39728	0.04530
0.45	0.000975	1.1	0.49	4.8%	30.08%	0.0121%	0.00001	0.00001	5.87552	0.06328	16.05%	0.0065%	0.00000	0.00000	3.05175	0.02431
0.56	0.000571	1.1	0.61	2.8%	40.02%	0.0068%	0.00001	0.00001	2.97709	0.03232	23.55%	0.0040%	0.00000	0.00000	1.81376	0.02124
0.67	0.000403	1.1	0.74	2.0%	45.77%	0.0077%	0.00001	0.00001	3.20794	0.05978	28.53%	0.0048%	0.00000	0.00000	2.11417	0.02357
0.78	0.000234	1.1	0.86	1.2%	53.18%	0.0051%	0.00000	0.00000	1.03638	0.01940	35.59%	0.0018%	0.00000	0.00000	0.74751	0.00809
0.89	0.000183	1.1	0.98	0.9%	57.41%	0.0051%	0.00000	0.00000	1.06844	0.02019	39.91%	0.0020%	0.00000	0.00000	0.80748	0.01578
1.01	0.000132	1.1	1.11	0.7%	62.86%	0.0051%	0.00001	0.00000	1.10208	0.03198	45.69%	0.0023%	0.00000	0.00000	0.88047	0.01673
1.12	0.000082	1.1	1.23	0.4%	65.99%	0.0015%	0.00000	0.00000	0.33861	0.00985	49.09%	0.0008%	0.00000	0.00000	0.27864	0.00524
1.23	0.000066	1.1	1.35	0.3%	70.06%	0.0015%	0.00000	0.00000	0.34379	0.01005	53.59%	0.0015%	0.00000	0.00000	0.29327	0.00874
1.34	0.000051	1.1	1.47	0.3%	72.42%	0.0015%	0.00000	0.00000	0.34630	0.01016	56.22%	0.0015%	0.00000	0.00000	0.30124	0.00889
1.45	0.000035	1.1	1.60	0.2%	74.55%	0.0007%	0.00000	0.00000	0.16765	0.00494	58.61%	0.0007%	0.00000	0.00000	0.14829	0.00435
1.56	0.000028	1.1	1.72	0.1%	77.35%	0.0007%	0.00000	0.00000	0.16871	0.00500	61.77%	0.0007%	0.00000	0.00000	0.15237	0.00444
1.68	0.000021	1.1	1.84	0.1%	79.00%	0.0003%	0.00000	0.00000	0.05731	0.00170	63.66%	0.0003%	0.00000	0.00000	0.05236	0.00152
1.79	0.000018	1.1	1.97	0.1%	81.20%	0.0003%	0.00000	0.00000	0.05752	0.00172	66.19%	0.0003%	0.00000	0.00000	0.05331	0.00155
1.90	0.000016	1.1	2.09	0.1%	82.51%	0.0003%	0.00000	0.00000	0.05762	0.00173	67.72%	0.0003%	0.00000	0.00000	0.05385	0.00156
2.01	0.000013	1.1	2.21	0.1%	84.26%	0.0003%	0.00000	0.00000	0.05775	0.00174	69.81%	0.0003%	0.00000	0.00000	0.05453	0.00158
2.12	0.000011	1.1	2.33	0.1%	85.31%	0.0003%	0.00000	0.00000	0.05781	0.00175	71.08%	0.0003%	0.00000	0.00000	0.05491	0.00160
2.23	0.000008	1.1	2.46	0.0%	86.73%	0.0000%	0.00000	0.00000	0.00099	0.00003	72.83%	0.0000%	0.00000	0.00000	0.00095	0.00003
Building Data					Σ Ann. MDF=	0.23564%	0.00007	0.00007	0.75546		Σ Ann. MDF=	0.10767%	0.00002	0.00002	0.32231	
Cost (\$CAN) = \$260/SF					Ann. Cost=	\$4,901	\$994	\$1,074	\$86	\$403	Ann. Cost=	\$2,240	\$244	\$315	\$33	\$172
Typical SF= 8000					50-Yr Cost=	\$245,069	\$49,697	\$53,719	\$4,298	\$20,146	50-Yr Cost=	\$111,978	\$12,184	\$15,750	\$1,672	\$8,595
Repl. Value= \$2,080,000					PV=	\$89,479	\$25,574	\$27,644	\$1,569	\$7,356	PV=	\$40,885	\$6,270	\$8,105	\$610	\$3,138
Full Upgrade Cost= \$33/SF					Savings= \$48,594											
Partial Upgrade Cost= \$10/SF					Total Savings= \$92,614											
Parapet Upgrade Cost= \$3/SF					Upgrade Cost= \$264,000											
Value of Life= \$9,100,000					B/C Ratio= 0.35											
Relocation Expense= \$2.90/SF																
Rental Cost= \$0.07/SF/day																
Occupancy Data																
Average Occupancy Density= .0036 Occ/SF																
Average No. of Occupants= 28.8																
Typical Streetfront Exposure= 30. lin. ft																
Average Pedestrian Density= .03 Ped./ft																
Average No. of Pedestrians= 0.9																
#of serious injuries/fatalities= 4																
%VSL assigned to injuries= 0.15																
					Economic Parameters											
					Time Horizon= 50 Years											
					Discount Rate for Capital= 5%											
					Discount Rate for Life-Safety= 3%											

Figure E.9 – Full Retrofit (Site Class D)

Cost-Benefit Analysis for Seismic Upgrades

Site Class= E		UNRETROFITTED										PARAPETS BRACED					
RGC (Class 'C')		Adjusted		UNRETROFITTED								PARAPETS BRACED					
Sa(1)	Ann. Exceedance	Fv	Sa(1)	%/50 Yrs	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	
0.01	0.150400	2.1	0.02	99.9%	0.14%	0.0093%	0.00000	0.00000	0.10247	0.00380	0.07%	0.0047%	0.00000	0.00000	0.00265	0.00027	
0.02	0.082400	2.1	0.05	98.4%	0.59%	0.0300%	0.00000	0.00000	2.18586	0.03295	0.40%	0.0202%	0.00000	0.00000	0.17890	0.00561	
0.06	0.031510	2.1	0.12	79.3%	4.15%	0.0456%	0.00000	0.00000	15.34719	0.09863	3.45%	0.0380%	0.00000	0.00000	4.52757	0.03845	
0.08	0.020510	2.1	0.16	64.1%	7.68%	0.0625%	0.00000	0.00001	27.55995	0.15412	6.63%	0.0540%	0.00000	0.00000	11.58536	0.07376	
0.11	0.012370	2.1	0.23	46.1%	13.13%	0.1102%	0.00001	0.00003	55.37675	0.29639	11.56%	0.0970%	0.00000	0.00001	30.81449	0.16631	
0.22	0.003974	2.0	0.45	18.0%	26.42%	0.0569%	0.00002	0.00003	28.28872	0.20959	23.36%	0.0503%	0.00001	0.00001	21.47708	0.15310	
0.34	0.001822	1.9	0.64	8.7%	40.02%	0.0339%	0.00003	0.00003	14.94547	0.16227	35.20%	0.0298%	0.00001	0.00001	13.08840	0.09318	
0.45	0.000975	1.7	0.76	4.8%	48.40%	0.0195%	0.00002	0.00002	7.89347	0.14701	42.59%	0.0172%	0.00001	0.00001	7.27802	0.07505	
0.56	0.000571	1.7	0.95	2.8%	55.36%	0.0169%	0.00001	0.00001	3.49975	0.06580	48.93%	0.0083%	0.00001	0.00001	3.32505	0.03565	
0.67	0.000403	1.7	1.14	2.0%	62.86%	0.0169%	0.00002	0.00001	3.66081	0.10622	56.05%	0.0169%	0.00001	0.00001	3.55739	0.06498	
0.78	0.000234	1.7	1.33	1.2%	68.78%	0.0051%	0.00001	0.00000	1.13001	0.03297	61.98%	0.0051%	0.00001	0.00000	1.11152	0.03170	
0.89	0.000183	1.7	1.52	0.9%	73.51%	0.0051%	0.00001	0.00000	1.14673	0.03373	66.97%	0.0051%	0.00001	0.00000	1.13568	0.03265	
1.01	0.000132	1.7	1.71	0.7%	77.35%	0.0051%	0.00001	0.00000	1.15710	0.03430	71.20%	0.0051%	0.00001	0.00000	1.15045	0.03340	
1.12	0.000082	1.7	1.90	0.4%	79.77%	0.0015%	0.00000	0.00000	0.35207	0.01049	73.97%	0.0015%	0.00000	0.00000	0.35069	0.01026	
1.23	0.000066	1.7	2.09	0.3%	82.51%	0.0015%	0.00000	0.00000	0.35353	0.01060	77.19%	0.0015%	0.00000	0.00000	0.35269	0.01041	
1.34	0.000051	1.7	2.28	0.3%	84.80%	0.0015%	0.00001	0.00000	0.35451	0.01069	79.97%	0.0015%	0.00000	0.00000	0.35400	0.01053	
1.45	0.000035	1.7	2.47	0.2%	86.73%	0.0007%	0.00000	0.00000	0.17097	0.00518	82.38%	0.0007%	0.00000	0.00000	0.17082	0.00511	
1.56	0.000028	1.7	2.66	0.1%	88.37%	0.0007%	0.00000	0.00000	0.17119	0.00520	84.46%	0.0007%	0.00000	0.00000	0.17111	0.00515	
1.68	0.000021	1.7	2.85	0.1%	89.43%	0.0003%	0.00000	0.00000	0.05801	0.00177	85.84%	0.0003%	0.00000	0.00000	0.05799	0.00175	
1.79	0.000018	1.7	3.04	0.1%	90.68%	0.0003%	0.00000	0.00000	0.05805	0.00177	87.47%	0.0003%	0.00000	0.00000	0.05804	0.00176	
1.90	0.000016	1.7	3.23	0.1%	90.68%	0.0003%	0.00000	0.00000	0.05805	0.00177	87.47%	0.0003%	0.00000	0.00000	0.05804	0.00176	
2.01	0.000013	1.7	3.42	0.1%	90.68%	0.0003%	0.00000	0.00000	0.05805	0.00177	87.47%	0.0003%	0.00000	0.00000	0.05804	0.00176	
2.12	0.000011	1.7	3.61	0.1%	93.10%	0.0003%	0.00000	0.00000	0.05811	0.00178	90.70%	0.0003%	0.00000	0.00000	0.05811	0.00178	
2.23	0.000008	1.7	3.80	0.0%	93.10%	0.0000%	0.00000	0.00000	0.00100	0.00003	90.70%	0.0000%	0.00000	0.00000	0.00100	0.00003	
Building Data					Σ Ann. MDF=	0.42432%	0.00018	0.00016		1.42884	Σ Ann. MDF=	0.35877%	0.00009	0.00008		0.85439	
Cost (\$CAN) = \$260/SF					Ann. Cost=	\$8,826	\$2,623	\$2,374	\$164	\$762	Ann. Cost=	\$7,462	\$1,274	\$1,198	\$101	\$456	
Typical SF= 8000					50-Yr Cost=	\$441,296	\$131,134	\$118,704	\$8,199	\$38,102	50-Yr Cost=	\$373,121	\$63,676	\$59,910	\$5,046	\$22,784	
Repl. Value= \$2,080,000					PV=	\$161,125	\$67,481	\$61,085	\$2,994	\$13,912	PV=	\$136,233	\$32,767	\$30,830	\$1,842	\$8,319	
Full Upgrade Cost= \$33/SF					Savings= \$24,892												
Partial Upgrade Cost= \$10/SF					\$34,714												
Parapet Upgrade Cost= \$3/SF					\$30,255												
Value of Life= \$9,100,000					\$1,151												
Relocation Expense= \$2.90/SF					\$5,593												
Rental Cost= \$0.07/SF/day					Total Savings= \$96,605												
					Upgrade Cost= \$24,000												
					B/C Ratio= 4.03												
Occupancy Data					Economic Parameters												
Average Occupancy Density= .0036 Occ/SF					Time Horizon= 50 Years												
Average No. of Occupants= 28.8					Discount Rate for Capital= 5%												
Typical Streetfront Exposure= 30. lin. ft					Discount Rate for Life-Safety= 3%												
Average Pedestrian Density= .03 Ped./ft																	
Average No. of Pedestrians= 0.9																	
#of serious injuries/fatalities= 4																	
%VSL assigned to injuries= 0.15																	

Figure E.10 – Parapet Bracing (Site Class E)

Cost-Benefit Analysis for Seismic Upgrades

Site Class= E						UNRETROFITTED						PARTIAL RETROFIT					
Sa(1)	RGC (Class 'C')	Adjusted Fv	Sa(1)	%/50 Yrs	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	
0.01	0.150400	2.1	0.02	99.9%	0.14%	0.0093%	0.00000	0.00000	0.10247	0.00380	0.05%	0.0036%	0.00000	0.00000	0.00159	0.00026	
0.02	0.082400	2.1	0.05	98.4%	0.59%	0.0300%	0.00000	0.00000	2.18586	0.03295	0.28%	0.0144%	0.00000	0.00000	0.11894	0.00540	
0.06	0.031510	2.1	0.12	79.3%	4.15%	0.0456%	0.00000	0.00000	15.34719	0.09863	2.58%	0.0284%	0.00000	0.00000	3.48276	0.03473	
0.08	0.020510	2.1	0.16	64.1%	7.68%	0.0625%	0.00000	0.00001	27.55995	0.15412	5.13%	0.0417%	0.00000	0.00000	9.38064	0.06559	
0.11	0.012370	2.1	0.23	46.1%	13.13%	0.1102%	0.00001	0.00003	55.37675	0.29639	9.28%	0.0779%	0.00000	0.00001	26.15641	0.14749	
0.22	0.003974	2.0	0.45	18.0%	26.42%	0.0569%	0.00002	0.00003	28.28872	0.20959	19.95%	0.0429%	0.00000	0.00001	19.45048	0.14124	
0.34	0.001822	1.9	0.64	8.7%	40.02%	0.0339%	0.00003	0.00003	14.94547	0.16227	31.26%	0.0265%	0.00001	0.00001	12.32591	0.08590	
0.45	0.000975	1.7	0.76	4.8%	48.40%	0.0195%	0.00002	0.00002	7.89347	0.14701	38.44%	0.0155%	0.00001	0.00001	6.97469	0.07072	
0.56	0.000571	1.7	0.95	2.8%	55.36%	0.0169%	0.00001	0.00001	3.49975	0.06580	44.64%	0.0075%	0.00000	0.00001	3.22433	0.03361	
0.67	0.000403	1.7	1.14	2.0%	62.86%	0.0169%	0.00002	0.00001	3.66081	0.10622	51.65%	0.0169%	0.00001	0.00001	3.48561	0.06281	
0.78	0.000234	1.7	1.33	1.2%	68.78%	0.0051%	0.00001	0.00000	1.13001	0.03297	57.54%	0.0051%	0.00000	0.00000	1.09630	0.03112	
0.89	0.000183	1.7	1.52	0.9%	73.51%	0.0051%	0.00001	0.00000	1.14673	0.03373	62.55%	0.0051%	0.00001	0.00000	1.12494	0.03212	
1.01	0.000132	1.7	1.71	0.7%	77.35%	0.0051%	0.00001	0.00000	1.15710	0.03430	66.85%	0.0051%	0.00001	0.00000	1.14283	0.03292	
1.12	0.000082	1.7	1.90	0.4%	79.77%	0.0015%	0.00000	0.00000	0.35207	0.01049	69.70%	0.0015%	0.00000	0.00000	0.34890	0.01012	
1.23	0.000066	1.7	2.09	0.3%	82.51%	0.0015%	0.00000	0.00000	0.35353	0.01060	73.07%	0.0015%	0.00000	0.00000	0.35140	0.01029	
1.34	0.000051	1.7	2.28	0.3%	84.80%	0.0015%	0.00001	0.00000	0.35451	0.01069	76.02%	0.0015%	0.00000	0.00000	0.35306	0.01042	
1.45	0.000035	1.7	2.47	0.2%	86.73%	0.0007%	0.00000	0.00000	0.17097	0.00518	78.60%	0.0007%	0.00000	0.00000	0.17049	0.00507	
1.56	0.000028	1.7	2.66	0.1%	88.37%	0.0007%	0.00000	0.00000	0.17119	0.00520	80.87%	0.0007%	0.00000	0.00000	0.17086	0.00511	
1.68	0.000021	1.7	2.85	0.1%	89.43%	0.0003%	0.00000	0.00000	0.05801	0.00177	82.40%	0.0003%	0.00000	0.00000	0.05792	0.00174	
1.79	0.000018	1.7	3.04	0.1%	90.68%	0.0003%	0.00000	0.00000	0.05805	0.00177	84.23%	0.0003%	0.00000	0.00000	0.05799	0.00175	
1.90	0.000016	1.7	3.23	0.1%	90.68%	0.0003%	0.00000	0.00000	0.05805	0.00177	84.23%	0.0003%	0.00000	0.00000	0.05799	0.00175	
2.01	0.000013	1.7	3.42	0.1%	90.68%	0.0003%	0.00000	0.00000	0.05805	0.00177	84.23%	0.0003%	0.00000	0.00000	0.05799	0.00175	
2.12	0.000011	1.7	3.61	0.1%	93.10%	0.0003%	0.00000	0.00000	0.05811	0.00178	87.93%	0.0003%	0.00000	0.00000	0.05808	0.00177	
2.23	0.000008	1.7	3.80	0.0%	93.10%	0.0000%	0.00000	0.00000	0.00100	0.00003	87.93%	0.0000%	0.00000	0.00000	0.00100	0.00003	
Building Data					Σ Ann. MDF=	0.42432%	0.00018	0.00016	1.42884		Σ Ann. MDF=	0.29802%	0.00006	0.00007	0.79373		
Cost (\$CAN) = \$260/SF					Ann. Cost=	\$8,826	\$2,623	\$2,374	\$164	\$762	Ann. Cost=	\$6,199	\$887	\$1,005	\$90	\$423	
Typical SF= 8000					50-Yr Cost=	\$441,296	\$131,134	\$118,704	\$8,199	\$38,102	50-Yr Cost=	\$309,945	\$44,356	\$50,274	\$4,483	\$21,166	
Repl. Value= \$2,080,000					PV=	\$161,125	\$67,481	\$61,085	\$2,994	\$13,912	PV=	\$113,167	\$22,826	\$25,871	\$1,637	\$7,728	
Full Upgrade Cost= \$33/SF																	
Partial Upgrade Cost= \$10/SF																	
Parapet Upgrade Cost= \$3/SF																	
Value of Life= \$9,100,000																	
Relocation Expense= \$2.90/SF																	
Rental Cost= \$0.07/SF/day																	
Occupancy Data																	
Average Occupancy Density= .0036 Occ/SF																	
Average No. of Occupants= 28.8																	
Typical Streetfront Exposure= 30. lin. ft																	
Average Pedestrian Density= .03 Ped./ft																	
Average No. of Pedestrians= 0.9																	
# of serious injuries/fatalities= 4																	
%VSL assigned to injuries= 0.15																	
Economic Parameters																	
Time Horizon= 50 Years																	
Discount Rate for Capital= 5%																	
Discount Rate for Life-Safety= 3%																	
											Savings=	\$47,959	\$44,656	\$35,214	\$1,357	\$6,184	
											Total Savings=	\$135,369					
											Upgrade Cost=	\$80,000					
											B/C Ratio=	1.69					

Figure E.11 – Partial Retrofit (Site Class E)

Cost-Benefit Analysis for Seismic Upgrades

Site Class= E																
Sa(1)	RGC (Class 'C')		Adjusted Sa(1)	% / 50 Yrs	UNRETROFITTED						RETROFITTED					
	Ann. Exceedance	Fv			MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days	MDF(Sa(1))	Ann. MDF	Indoor Fatality	Outdoor Fatality	Ann. Reloc. Exp.	Ann. Lost Rnt'l Days
0.01	0.150400	2.1	0.02	99.9%	0.14%	0.0093%	0.00000	0.00000	0.10247	0.00380	0.04%	0.0025%	0.00000	0.00000	0.01107	0.00090
0.02	0.082400	2.1	0.05	98.4%	0.59%	0.0300%	0.00000	0.00000	2.18586	0.03295	0.20%	0.0101%	0.00000	0.00000	0.29941	0.00904
0.06	0.031510	2.1	0.12	79.3%	4.15%	0.0456%	0.00000	0.00000	15.34719	0.09863	1.69%	0.0186%	0.00000	0.00000	3.43725	0.03286
0.08	0.020510	2.1	0.16	64.1%	7.68%	0.0625%	0.00000	0.00001	27.55995	0.15412	3.34%	0.0272%	0.00000	0.00000	7.66256	0.05540
0.11	0.012370	2.1	0.23	46.1%	13.13%	0.1102%	0.00001	0.00003	55.37675	0.29639	6.07%	0.0509%	0.00000	0.00001	19.17756	0.11576
0.22	0.003974	2.0	0.45	18.0%	26.42%	0.0569%	0.00002	0.00003	28.28872	0.20959	13.64%	0.0294%	0.00000	0.00001	13.71467	0.11512
0.34	0.001822	1.9	0.64	8.7%	40.02%	0.0339%	0.00003	0.00003	14.94547	0.16227	23.55%	0.0199%	0.00001	0.00001	9.10536	0.10662
0.45	0.000975	1.7	0.76	4.8%	48.40%	0.0195%	0.00002	0.00002	7.89347	0.14701	30.95%	0.0125%	0.00001	0.00001	5.37806	0.05912
0.56	0.000571	1.7	0.95	2.8%	55.36%	0.0169%	0.00001	0.00001	3.49975	0.06580	37.79%	0.0064%	0.00000	0.00000	2.58684	0.05128
0.67	0.000403	1.7	1.14	2.0%	62.86%	0.0169%	0.00002	0.00001	3.66081	0.10622	45.69%	0.0077%	0.00001	0.00001	2.92469	0.05558
0.78	0.000234	1.7	1.33	1.2%	68.78%	0.0051%	0.00001	0.00000	1.13001	0.03297	52.17%	0.0051%	0.00000	0.00000	0.95338	0.02858
0.89	0.000183	1.7	1.52	0.9%	73.51%	0.0051%	0.00001	0.00000	1.14673	0.03373	57.44%	0.0051%	0.00000	0.00000	1.00621	0.02960
1.01	0.000132	1.7	1.71	0.7%	77.35%	0.0051%	0.00001	0.00000	1.15710	0.03430	61.77%	0.0051%	0.00000	0.00000	1.04499	0.03046
1.12	0.000082	1.7	1.90	0.4%	79.77%	0.0015%	0.00000	0.00000	0.35207	0.01049	64.54%	0.0015%	0.00000	0.00000	0.32331	0.00939
1.23	0.000066	1.7	2.09	0.3%	82.51%	0.0015%	0.00000	0.00000	0.35353	0.01060	67.72%	0.0015%	0.00000	0.00000	0.33037	0.00959
1.34	0.000051	1.7	2.28	0.3%	84.80%	0.0015%	0.00001	0.00000	0.35451	0.01069	70.45%	0.0015%	0.00000	0.00000	0.33573	0.00976
1.45	0.000035	1.7	2.47	0.2%	86.73%	0.0007%	0.00000	0.00000	0.17097	0.00518	72.83%	0.0007%	0.00000	0.00000	0.16360	0.00477
1.56	0.000028	1.7	2.66	0.1%	88.37%	0.0007%	0.00000	0.00000	0.17119	0.00520	74.93%	0.0007%	0.00000	0.00000	0.16514	0.00483
1.68	0.000021	1.7	2.85	0.1%	89.43%	0.0003%	0.00000	0.00000	0.05801	0.00177	76.35%	0.0003%	0.00000	0.00000	0.05623	0.00165
1.79	0.000018	1.7	3.04	0.1%	90.68%	0.0003%	0.00000	0.00000	0.05805	0.00177	78.08%	0.0003%	0.00000	0.00000	0.05658	0.00166
1.90	0.000016	1.7	3.23	0.1%	90.68%	0.0003%	0.00000	0.00000	0.05805	0.00177	78.08%	0.0003%	0.00000	0.00000	0.05658	0.00166
2.01	0.000013	1.7	3.42	0.1%	90.68%	0.0003%	0.00000	0.00000	0.05805	0.00177	78.08%	0.0003%	0.00000	0.00000	0.05658	0.00166
2.12	0.000011	1.7	3.61	0.1%	93.10%	0.0003%	0.00000	0.00000	0.05811	0.00178	81.70%	0.0003%	0.00000	0.00000	0.05717	0.00170
2.23	0.000008	1.7	3.80	0.0%	93.10%	0.0000%	0.00000	0.00000	0.00100	0.00003	81.70%	0.0000%	0.00000	0.00000	0.00098	0.00003
Building Data					Σ Ann. MDF=	0.42432%	0.00018	0.00016	1.42884		Σ Ann. MDF=	0.20786%	0.00005	0.00005	0.73704	
Cost (\$CAN) = \$260/SF					Ann. Cost=	\$8,826	\$2,623	\$2,374	\$164	\$762	Ann. Cost=	\$4,324	\$734	\$763	\$69	\$393
Typical SF= 8000					50-Yr Cost=	\$441,296	\$131,134	\$118,704	\$8,199	\$38,102	50-Yr Cost=	\$216,175	\$36,702	\$38,143	\$3,445	\$19,655
Repl. Value= \$2,080,000					PV=	\$161,125	\$67,481	\$61,085	\$2,994	\$13,912	PV=	\$78,930	\$18,887	\$19,628	\$1,258	\$7,176
Full Upgrade Cost= \$33/SF					Savings= \$82,196											
Partial Upgrade Cost= \$10/SF					Total Savings= \$180,718											
Parapet Upgrade Cost= \$3/SF					Upgrade Cost= \$264,000											
Value of Life= \$9,100,000					B/C Ratio= 0.68											
Relocation Expense= \$2.90/SF																
Rental Cost= \$0.07/SF/day																
Occupancy Data					Economic Parameters											
Average Occupancy Density= .0036 Occ/SF					Time Horizon= 50 Years											
Average No. of Occupants= 28.8					Discount Rate for Capital= 5%											
Typical Streetfront Exposure= 30. lin. ft					Discount Rate for Life-Safety= 3%											
Average Pedestrian Density= .03 Ped./ft																
Average No. of Pedestrians= 0.9																
# of serious injuries/fatalities= 4																
%VSL assigned to injuries= 0.15																

Figure E.12 – Full Retrofit (Site Class E)

Appendix F

Data Collection Forms

As discussed in Chapter 7, data collection forms were specifically developed for a pilot survey of URM buildings in Victoria. Four sheets were included: Exterior Access, Interior Access, Roof Access, and a Reference sheet. Organizing the data collection by access level allowed for varied access, depending upon owners' level of comfort. In most cases, only exterior access (i.e. from the sidewalk) was achieved. Towards the completion of this study, the sheets were revised, primarily to incorporate the score modifiers for the refined FEMA 154 screening methodology. The final forms are provided on the following pages.

Appendix F – Data Collection Forms

Building Field Data Collection Form
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UBC Study:
URM Buildings Survey in Victoria, BC

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PART 1 - EXTERIOR ACCESS

SKETCH OF OVERALL BUILDING			BASIC INFORMATION					
<p>Plan View Photo/Sketch (show/label the adjacent streets)</p>			Historic Address: _____ Bldg Name: _____ Year Built: _____ Postal Code: _____ Latitude: _____ Longitude: _____ Bldg Owner: _____ Site Class: _____ S _s (0.2): _____ Height [ft]: _____ Storeys Above Gr: _____ NZ Bldg Type: _____ Est. Flr Area (ft ²): _____ Other Address(es): _____					
			GENERAL EXTERIOR					
<p>Elevation View(s) Photo/Sketch (note which street viewed from)</p>			Open front? (Y/N-show on elev sketch, eg. mostly glass): _____ LFRS @ open front? (steel braces, moment frame): _____ Adacent Buildings? (Y/N-dimension elev. sketch): _____ Shares wall w/ adj bldg? (Y/N-show on elev. sketch): _____ Adj bldg # storeys dif.? (Y/N-show on elev. sketch): _____ Unmatching Floors? (Y/N-show on elev. sketch): _____ End building? (Y/N-show on plan sketch): _____ Header courses every _____ courses Veneer? (Y/N, eg. weep holes, no header courses): _____					
			OCCUPANCY (circle all that apply)					
			High Density	Medium Density		Low Density		
			Assembly	Offices	Residential	Storage		
			Mercantile	Institutional	Hotel			
			Personal Service	Manufacturing	Industrial			
			Restaurant/Bar					
			EXTERIOR AREAS					
			Lower Roofs of Adjacent Bldgs: _____ Sidewalks, Alleys? (Y/N-show on plan sketch): _____ Seating Area? (Y/N-show on plan sketch, eg. patio): _____					
			MASONRY CONDITION					
			Scratch Test Trial (show locations on sketch)					
				Trial 1	Trial 2	Trial 3		
			Brick					
			Mortar					
			Max. crack width? [mm]: _____					
			Areas of repointed mortar? (Y/N-eg. dif colors): _____ Comments (eg. raked joints): _____					
			EXT. WALL HEIGHT/THICKNESS (use interior instead if possible)					
				1st Storey	Typ. Storey	Top Storey		
			Height (ft)					
			Width (in)					
			SIGNS OF SEISMIC RETROFIT (use interior instead if possible)					
			Anchor plates visible @ floor levels? (Y/N): _____ Steel braces visible in windows? (Y/N): _____					
			GENERAL COMMENTS					
			Note items such as: additions (eg. wood storey); structural deterioration not noted above					
FEMA 154 SCORING			FALL HAZARDS (use roof access instead if possible)					
Basic Score (URM)	1.8		Type	Parapets	Cornices (Heavy)	Cornices (Light)	Chimneys	
Vertical Irreg.	-1.0		Present (Y/N)					
Plan Irregularity	-0.5		Elev. (N,S,E,W)					
Soil Type (Circle One)	A/B -- C -0.4 D -0.6 E/F -0.8		Height (in)					
			Braced?					
			Areas Below (1,2,3)					
Bldg Vuln. (Circle one)	High -0.1 Mod. -- Low +0.1		Other fall hazards (describe):					
Ped. Density (Circle one)	High -0.3 Mod. -- Low +0.3		Notes: - Cornices: wood/tin = light; terracotta/brick = heavy - Show fall hazards on sketches - For "Braced?": indicate Some/All/DNK - For "Areas Below": 1=sidewalk/alley, 2=adj. bldg, 3=low traffic					
Occupant Density (Circle one)	High/Mod. -0.15 Mod. -- Mod./Low +0.15 Low +0.3		FALL HAZARDS COMMENTS					
			Note items such as deterioration, damage, mitigation work in progress					
Retrofit Extents (Circle one)	None -- Part'l Para. +0.2 All Para. +0.6 Tens. Ties +0.7 Full +1.0							
FINAL SCORE, S =								

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Building Field Data Collection Form
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UBC Study:
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PART 2 - INTERIOR ACCESS

GRAVITY SYSTEM (circle all that apply)			GENERAL INTERIOR INFORMATION						
Exterior URM Walls	Light Wood Frame Walls	Dim. Lumber Joists	Owner Rep: _____		Tel./email: _____				
Concrete Columns	Heavy Timber Columns	Steel Columns	# Occupants (day): _____		# Occ. (night): _____				
Concrete Beams	Heavy Timber Beams	Steel Beams	Basement Use? (eg. storage, N/A): _____						
Others: (describe): _____			1st Floor Use? (eg. retail, offices): _____						
FOUNDATIONS			2nd Floor Use? (eg. residential, unused): _____						
Concrete	Stone/Rubble	Other	3rd Floor Use? (eg. residential, unused): _____						
RETROFIT EXTENTS			4th Floor Use? (eg. residential, unused): _____						
Any seismic upgrades? (Y/N; ask owner - if yes, complete items below): _____			# Storeys below grade (eg. ½, 1, 1½): _____						
What year were upgrades completed? (ask owner): _____			Basement extends below sidewalk?(Y/N - show where on sketch): _____						
Walls, In-plane? (Y/N - eg. steel braces/moment frame): _____			Is building sprinklered? (Y/N - i.e. are sprinkler heads visible?): _____						
Walls, Out-of-plane? (Y/N - eg. vertical steel "strongbacks"): _____			EXTERIOR WALL HEIGHT/THICKNESS & LAY-UP						
Diaphragm? (Y/N - eg. plywood atop old, steel angles at perimeter): _____			NOTE: mark with cross (†) if VENEER wythe is present						
Diaphragm-wall conn.? (Y/N - eg. epoxy anchors, plates on exterior): _____			1st Storey measurements (label walls on sketch)						
PHOTO CHECKLIST				Wall 1	Wall 2	Wall 3			
Diaphragm (floor) <input type="checkbox"/>	Wall-Floor Connection <input type="checkbox"/>	Col.-Beam Conn. <input type="checkbox"/>	Wall 4						
SKETCH OF FLOOR PLAN			height[ft]						
<i>Just a rough outline for wall labels; show the adjacent streets on the sketch</i>			width [in]						
<div style="border: 1px solid black; width: 100%; height: 100%; background-image: linear-gradient(to right, transparent 49%, #ccc 49% 51%, #ccc 51% 53%, transparent 53%); background-size: 20px 20px;"></div>			# wythes						
			Header courses typically every _____ courses						
			2nd Storey measurements (label walls on sketch)				Wall 1	Wall 2	Wall 3
			height[ft]			Wall 4			
			width [in]						
			# wythes						
			Header courses typically every _____ courses						
			3rd Storey measurements (label walls on sketch)				Wall 1	Wall 2	Wall 3
			height[ft]			Wall 4			
			width [in]						
# wythes									
Header courses typically every _____ courses									
4th Storey measurements (label walls on sketch)				Wall 1	Wall 2	Wall 3			
height[ft]			Wall 4						
width [in]									
# wythes									
Header courses typically every _____ courses									
GENERAL COMMENTS									
eg: "many tall/heavy bldg contents on flr X" "Basement under sidewalk retrofitted"									

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Appendix F – Data Collection Forms

Building Field Data Collection Form
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UBC Study:
URM Buildings Survey in Victoria, BC

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PART 3 - ROOF ACCESS

SKETCH OF ROOF	PARAPET/CORNICE INFORMATION																				
<p style="font-size: small; color: grey;"><i>Show the adjacent streets, and along which streets parapets, etc. are present</i></p> <div style="border: 1px solid grey; height: 150px; width: 100%;"></div>	<p style="font-size: small; color: grey;"><i>Height & Thickness Measurements (label walls on roof sketch)</i></p> <table border="1" style="width: 100%; border-collapse: collapse; font-size: x-small;"> <thead> <tr style="background-color: #cccccc;"> <th></th> <th>Wall 1</th> <th>Wall 2</th> <th>Wall 3</th> <th>Wall 4</th> </tr> </thead> <tbody> <tr> <td>Height [ft]</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Width [in]</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td># Wythes</td> <td></td> <td></td> <td></td> <td></td> </tr> </tbody> </table> <p>Parapets/Cornices present? (Y/N - show extents on roof sketch): _____</p> <p>Bracing present? (Y/N - show on parapet sketch): _____</p> <p>Approx. spacing of braces? (note on parapet sketch): _____ [ft]</p> <p>Continuous ledger member? (Y/N - show on parapet sketch): _____</p> <p>Replaced with new materials? (Y/N - if Y, complete next item): _____</p> <p>Describe new mat'l's (eg. sheet steel): _____</p> <p>Max. # of courses between headers: _____</p>		Wall 1	Wall 2	Wall 3	Wall 4	Height [ft]					Width [in]					# Wythes				
	Wall 1	Wall 2	Wall 3	Wall 4																	
Height [ft]																					
Width [in]																					
# Wythes																					
SKETCH OF TYPICAL PARAPET	ROOFING INFORMATION																				
<p style="font-size: small; color: grey;"><i>Show rough shape, height from roof and width, other relevant observations</i></p> <div style="border: 1px solid grey; height: 150px; width: 100%;"></div>	<p>Type (eg. torch-on, tar and gravel, sheet metal): _____</p> <p>What year was roof last replaced? (ask building owner): _____</p> <p>Condition (good, fair, poor): _____</p> <p>Com'ts (eg. blisters, cracking): _____</p>																				
SKETCH OF TYPICAL PARAPET	MASONRY CONDITION																				
<p style="font-size: small; color: grey;"><i>Show rough shape, height from roof and width, other relevant observations</i></p> <div style="border: 1px solid grey; height: 150px; width: 100%;"></div>	<p style="font-size: small; color: grey;"><i>Scratch Test Trial (show locations on roof sketch)</i></p> <table border="1" style="width: 100%; border-collapse: collapse; font-size: x-small;"> <thead> <tr style="background-color: #cccccc;"> <th></th> <th>Wall 1</th> <th>Wall 2</th> <th>Wall 3</th> <th>Wall 4</th> </tr> </thead> <tbody> <tr> <td>Brick</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Mortar</td> <td></td> <td></td> <td></td> <td></td> </tr> </tbody> </table> <p>Maximum crack width? [mm]: _____</p> <p>Com'ts (eg. eroded mortar): _____</p>		Wall 1	Wall 2	Wall 3	Wall 4	Brick					Mortar									
	Wall 1	Wall 2	Wall 3	Wall 4																	
Brick																					
Mortar																					
SKETCH OF TYPICAL CHIMNEYS/PILASTER	CHIMNEY/PILASTER INFORMATION																				
<p style="font-size: small; color: grey;"><i>Show rough shape, height from roof and least width</i></p> <div style="border: 1px solid grey; height: 150px; width: 100%;"></div>	<p style="font-size: small; color: grey;"><i>Height & Thickness Measurements (label on roof sketch)</i></p> <table border="1" style="width: 100%; border-collapse: collapse; font-size: x-small;"> <thead> <tr style="background-color: #cccccc;"> <th></th> <th>Chimney 1</th> <th>Chimney 2</th> <th>Chimney 3</th> <th>Chimney 4</th> </tr> </thead> <tbody> <tr> <td>Height [ft]</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>Width [in]</td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td># Wythes</td> <td></td> <td></td> <td></td> <td></td> </tr> </tbody> </table> <p>Chimeys present? (show locations on roof sketch): _____</p> <p>Bracing present? (Y/N - show on chimney sketch): _____</p> <p>Replaced with new materials? (Y/N - if Y, complete next item): _____</p> <p>Describe new mat'l's (eg. sheet steel): _____</p> <p>Max. # of courses between headers: _____</p> <p>Com'ts (eg. corroded anchors): _____</p>		Chimney 1	Chimney 2	Chimney 3	Chimney 4	Height [ft]					Width [in]					# Wythes				
	Chimney 1	Chimney 2	Chimney 3	Chimney 4																	
Height [ft]																					
Width [in]																					
# Wythes																					
SKETCH OF TYPICAL CHIMNEYS/PILASTER	SKETCH OF TYPICAL CORNICE																				
<p style="font-size: small; color: grey;"><i>Show rough shape, height from roof and least width</i></p> <div style="border: 1px solid grey; height: 150px; width: 100%;"></div>	<p style="font-size: small; color: grey;"><i>Show rough shape, approx. height from roof and overhang</i></p> <div style="border: 1px solid grey; height: 150px; width: 100%;"></div>																				

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Appendix F – Data Collection Forms

Building Field Data Collection Form
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URM Study:
URM Buildings Survey in Victoria, BC

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REFERENCE SHEET

VERTICAL IRREGULARITIES (Source: FEMA 454)			
vertical conditions	resulting failure patterns	performance	code remedies
		V1 Stiffness Irregularity: Soft Story Common collapse mechanism. Death and much damage in Northridge earthquake.	Modal Analysis, +6.5 feet high in SCD D,E,F. Extreme case not permitted in seismic use groups E and F.
		V2 Weight/Mass Irregularity Collapse mechanism in extreme circumstances.	Modal Analysis, +6.5 foot high in SDC D,E,F.
		V3 Vertical Geometric Irregularity Localized structural damage.	Modal Analysis, +6.5 foot high in SDC D,E,F.
		V4 In-Plane Irregularity in Vertical Lateral Force System Localized structural damage.	Modal Analysis, +6.5 foot high in SDC D, E, F. 25% increase to diaphragm connection design force. Supporting members designed for increased forces.
		V5 Capacity Discontinuity: Weak Story Collapse mechanism in extreme circumstances	Modal Analysis, +6.5 foot high in SDC D,E,F.

SCRATCH TEST		
<i>Brick Scratch Test Indices</i>		
Description	Index	
Scratches using aluminum	2.5	
Scratches using copper penny	3.0	
Does not scratch w/ above tools	4.0	
<i>Mortar Scratch Test Indices</i>		
Description	Index	
Scratches using fingernail	1.5	
Scratches using copper penny	2.0	
Does not scratch w/ above tools	2.5	
NZ Bldg Typology		
Category	Description	Vuln.
A	1 St, Isolated	Low
B	1 St, Row	Low
C	2 St, Isolated	High
D	2 St, Row	Moderate
E	3 St, Isolated	High
F	3 St, Row	Moderate
G	Indus/Relig.	High

PLAN IRREGULARITIES (Source: FEMA 454)			
plan conditions	resulting failure patterns	performance	code remedies
		P1 Torsional Irregularity: Unbalanced Resistance Localized damage. Collapse mechanism in extreme instances.	Modal Analysis, +6.5 foot high in SDC D, E, F. 25% increase to diaphragm connection design forces. Amplified forces to max of X3.
		P2 Re-entrant Corners Local damage to diaphragm and attached elements. Collapse mechanism in extreme instances in large buildings.	25% increase in diaphragm connection design forces.
		P3 Diaphragm Eccentricity and Cutouts Localized structural damage.	25% increase in diaphragm connection design forces.
		P4 Nonparallel Lateral Force-Resisting System Leads to torsion and instability, localised damage.	Combine 100% and 30% of forces in 2 directions, use maximum.
		P5 Out-of-Plane Offsets: Discontinuous Shearwalls Collapse mechanism in extreme circumstances.	Modal Analysis, +6.5 foot high in SDC D,E,F. 25% increase to diaphragm connection design forces.

MASONRY TERMINOLOGY		
		WYTHE
		COURSE
		STRETCHER HEADER
		SOLDIER

ROOF TYPE EXAMPLES			
Torch-on 	Tar & Gravel 	Inverted Roof (rocks deeper than "Tar & Gravel") 	Sheet Metal

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