Seismic Risk Assessment of Unreinforced Masonry Buildings Using Fuzzy Based Techniques for the Regional Seismic Risk Assessment of Ottawa, Ontario

By

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Abstract

Unreinforced masonry construction is considered to be the most vulnerable forms of construction as demonstrated through recent earthquakes. In Canada, many densely populated cities such as (Vancouver, Montreal and Ottawa) have large inventories of seismically vulnerable masonry structures. Although measures have been taken to rehabilitate and increase the seismic resistance of important and historic structures, many existing unreinforced masonry structures have not been retrofitted and remain at risk in the event of a large magnitude earthquake. There is therefore a need to identify buildings at risk and develop tools for assessing the seismic vulnerability of existing unreinforced masonry structures in Canada.

This thesis presents results from an ongoing research program which forms part of a multi-disciplinary effort between the University of Ottawa's Hazard Mitigation and Disaster Management Research Centre and the Geological Survey of Canada (NRCAN) to assess the seismic vulnerability of buildings in dense urban areas such as Ottawa, Ontario. A risk-based seismic assessment tool (CanRisk) has been developed to assess the seismic vulnerability of existing unreinforced masonry and reinforced concrete structures. The seismic risk assessment tool exploits the use of fuzzy logic, a soft computing technique, to capture the vagueness and uncertainty within the evaluation of the performance of a given building. In order to conduct seismic risk assessments, a general building inventory and its spatial distribution and variability is required for earthquake loss estimations. The Urban Rapid Assessment Tool (Urban RAT) is designed for the rapid collection of building data in urban centres. This Geographic Information System (GIS) based assessment tool allows for intense data collection and revolutionizes the traditional sidewalk survey approach for collecting building data. The application of CanRisk and the Urban RAT tool to the City of Ottawa is discussed in the following thesis. Data collection of over 13,000 buildings has been obtained including the seismic risk assessment of 1,465 unreinforced masonry buildings. A case study of selected URM buildings located in the City of Ottawa was conducted using CanRisk. Data obtained from the 2011 Christchurch Earthquake in New Zealand was utilized for verification of the tool.

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This thesis is dedicated to my family who have provided me with unconditional love and a strong foundation to achieve all of that I have in life.

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Nomenclature

%NBS	Percentage New Building Standard
A_i	Area of a membership function (i)
A_{i1}, A_{i2}	Input membership fuzzy sets for i^{th} rule
B	Output membership fuzzy set
C_{i_x}	Centroid of membership function (i) along the x-axis
C1	Concrete moment frame buildings
C2	Concrete shear wall buildings
C3	Concrete frames with infill masonry shear walls
h_n	Building height in meters
I^{BD}	Building Damageability Index
I_{NS}^{BD}	Non-structural building damageability index
I_S^{BD}	Structural building damageability index
I^{BV}	Building Vulnerability Index
I_{NS}^{BV}	Non-structural building vulnerability index
I_S^{BV}	Structural building vulnerability index
I_E	Building importance factor as defined in NBCC-2010
I^{IE}	Building Importance/Exposure Index
I^R	Risk Index
I_{NS}^R	Non-structural risk index
I_S^R	Structural risk index
I_{ALL}^R	Overall building risk index
I^{SSH}	Site Seismic Hazard Index
K	Total number of rules in a FRB
M_v	Higher mode effect factor according to NBCC-2005
N	Number of stories
R_d	Ductility-related force modification factor, as specified in the National Building Code of Canada
R_i	Represents the i^{th} rule of a FRB
R_o	Overstrength-related force modification factor, as specified in the National Building Code of Canada

Sa	Spectral acceleration
Sa(T1)	Spectral acceleration (% of g) for a building with a period of T1
$S_a^c(T_1)$	Spectral acceleration (% of g) for a building with period T1 on soil class C
T1	Fundamental period of vibration of a building
URM	Unreinforced masonry buildings
V_s	Shear wave velocity
W	Weight of a building
X_1, X_2	Input variables (antecedents)
Y	Output crisp variable (consequent)
μ_i^j	Fuzzy membership function (i) for parameter (j)
$(\mu_{VL}^{ID}, \mu_L^{ID}, \mu_M^{ID}, \mu_H^{ID}, \mu_{VH}^{ID})$	
	Fuzzy membership function for *VL, L, M, H* and *VH* of increase in demand
$(\mu_{VL}^{VI}, \mu_L^{VI}, \mu_M^{VI}, \mu_H^{VI}, \mu_{VH}^{VI})$	
	Fuzzy membership function for *VL, L, M, H* and *VH* of vertical irregularity
$(\mu_{VL}^{PI}, \mu_L^{PI}, \mu_M^{PI}, \mu_H^{PI}, \mu_{VH}^{PI})$	
	Fuzzy membership function for *VL, L, M, H* and *VH* of plan irregularity

Acronyms

AP	Apartment
ASCE	American Society of Civil Engineering
ATC	American Technology Council
AVG	Average
BD	Building damageability
BI/E	Building importance/exposure
BIF	Building importance factor
BU	Building use
BV	Building vulnerability
BSS	Building structural system
C	Church
CBD	Central Business District
CC	Catastrophic consequences
CEN	European Committee for Standardization
COL	At/Near collapse
CRIT	Critical
CSA	Canadian Standard Association
CSV	Comma separated values
CQ	Construction quality
D	Dwelling
DDF	Design and detailing factor
DR	Decrease in resistance
EERI	Earthquake Engineering Research Institute
EI	Economic impact
F	Food (store)
FEMA	Federal Emergency Management Agency
FHL	Falling hazards to life
FIS	Fuzzy inference system
FRB	Fuzzy rule base

FSE	Fuzzy synthetic evaluation
GD	Good
GIS	Geographical information system
GPS	Global positioning system
H	High
HAZUS-MH	Hazards U.S. – Multi-Hazards
HV	Heavy
HVO	Hazards to vital operations
IBC	International Building Code
ID	Increase in demand
IEP	Initial evaluation procedure
INSH	Increase in non-structural hazard
L	Low
LFRS	Later force-resisting system
LT	Light
M	Moderate
MARG	Marginal
N/A	Not Available
NBCC	National Building Code of Canada
NEHRP	National Earthquake Hazards Reduction Program
NG	Negligible
NIBS	National Institute of Building Sciences
NRCC	National Research Council of Canada
NSBD	Non-structural building damageability
NSBV	Non-structural building vulnerability
NSD	Non-structural deficiency
NSR	Non-structural risk
NSV	Non-structural vulnerability
NZ	New Zealand
NZSEE	New Zealand Society for Earthquake Engineering
OBR	Overall building risk

PGA	Peak ground acceleration
PR	Poor
R	Retail
RC	Reinforced concrete
RVS	Rapid visual screening
SBD	Structural building damageability
SBV	Structural building vulnerability
SFF	Seismic force factor
SFRS	Seismic force-resisting system
SPI	Seismic priority index
SR	Structural risk
SS	Soft story
SSH	Site seismic hazard
SV	Structural vulnerability
TFN	Triangular fuzzy numbers
TI	Torsional irregularity
TPFN	Trapezoidal fuzzy numbers
U	Unknown
UBC	Uniform Building Code
UHS	Uniform hazard spectrum
URBAN RAT	Urban Rapid Assessment Tool
URM	Unreinforced Masonry Buildings
VH	Very high
VI	Vertical irregularity
VL	Very low
WS	Weak story
XML	Extensible mark-up language
YOC	Year of construction

Chapter 1
Introduction

1.1 General

The 1868 Hayward, 1906 San Francisco, 1925 Santa Barbara, and 1933 Long Beach earthquakes exposed in dramatic fashion the consequences of poorly designed masonry structures (ATC, 2009). Following the damage observed in the Long Beach earthquake (see example in Figure 1-1), building codes in California prohibited the construction of unreinforced masonry (URM) buildings (Hess, 2008). Despite this fact, and regardless of some rehabilitation efforts, many existing masonry structures constructed prior to this era remained vulnerable. The 1971 San Fernando, 1989 Loma Prieta and 1994 Northridge earthquakes exposed the urgent need to identify and retrofit vulnerable structures. Similarly, in New Zealand, the 2010 Darfield and 2011 Christchurch earthquakes also exposed the seismic risk associated with poorly designed masonry structures (Ingham amd Griffith, 2011a).

Figure 1-1: Complete Collapse of a Commercial URM building, Long beach, California 1933

(Hess, 2008)

Similarly, in Canada, many densely populated cities (Vancouver, Montreal and Ottawa) have large inventories of unreinforced masonry structures (Hodgson, 1945; Bruneau and Lamontgne 1994). With approximately 40% of Canadians living in areas of high or moderate earthquake risk, it is essential to understand the potential hazards posed by vulnerable URM buildings (Statistics Canada, 2011; Kovacs, 2010; Bruneau 1994). In the Ottawa-Gatineau region, continuous urban growth puts ever greater populations and infrastructure at risk to seismic disturbance (Lamontagne, 2010). Although measures have been taken to rehabilitate and increase the seismic resilience of important and historic structures, many existing URM structures have not been retrofitted and remain at risk in the event of a large magnitude earthquake. There is therefore a need to identify vulnerable structures and develop tools for assessing the seismic vulnerability of masonry structures in Canada.

Due to large inventories of URM buildings and the hazard associated with this structural type, it is essential to include the evaluation of URM buildings when conducting seismic risk assessments (Bruneau 1994; Mitchell et al. 1990). Seismic risk assessments provide knowledge to support effective actions by decision makers that can reduce potential damage to populated urban communities. In the case of seismically deficient URM buildings, information gathered from risk assessments can provide insight on potential mitigation techniques (retrofit, demolition, etc.).

The work presented in this thesis forms part of a multi-disciplinary effort between the University of Ottawa's Hazard Mitigation and Disaster Management Research Centre and the Geological Survey of Canada (NRCAN) to assess the seismic vulnerability of buildings in dense urban areas such as Ottawa, Ontario.

A risk-based seismic assessment tool (CanRisk), that can be used to assess the seismic vulnerability of existing unreinforced masonry and reinforced concrete buildings, is presented. The tool exploits the use of fuzzy logic, a soft computing technique, to capture the vagueness and uncertainty within the evaluation of the performance of a given building.

In order to conduct seismic risk assessment of a geographic region, a general building inventory and its spatial distribution and vulnerability is required for earthquake loss estimations. The thesis presents the application of Urban Rapid Assessment Tool (Urban

RAT), developed at the University of Ottawa as a tool for rapid collection of building data in urban centres. This Geographic Information System (GIS) based assessment tool allows for intense data collection while revolutionizing the traditional sidewalk survey approach for collecting building data.

1.2 Research Objective and Scope

The main objective of this thesis is to enhance and expand the application of CanRisk, an existing risk-based seismic assessment tool developed by Tesfamariam (2008) at the University of Ottawa for reinforced concrete buildings. In particular, the tool is extended to allow for the seismic risk assessment of URM buildings. Furthermore, the tool is enhanced by introducing new performance modifiers that take into account the effects of structural and non-structural vulnerabilities to compute overall building risk. The objective is achieved by utilizing the current framework of the existing version of CanRisk (Tesfamariam, 2008) and modifying it to increase the program's functionality.

In addition, contributions are made towards the development of a new GIS-based tool (Urban RAT), which is used for rapid visual screening (RVS) of buildings in the City of Ottawa to establish a building database. A total of over 13,000 buildings are assessed in Ottawa for seismic risk assessment.

CanRisk and Urban RAT are used to conduct seismic risk assessment of 1,465 unreinforced masonry and 580 reinforced concrete buildings. A case study of a selected number of URM buildings located in the City of Ottawa are evaluated using CanRisk. The data obtained from the 2011 Christchurch earthquake in New Zealand is utilized to verify the capabilities of CanRisk as a seismic risk assessment tool.

1.3 Thesis Structure

As demonstrated in Figure 1-2 , the thesis consists of seven sections:

- Chapter 1: Introduces the thesis and the research objective and scope;
- Chapter 2: Presents a literature review on the thesis subject matter, including a review of seismic risk, building data collection, seismic risk assessment methods and

fuzzy logic theory and presents a State-of-the-art literature review on the seismic vulnerability of unreinforced masonry buildings;

- Chapter 3: Presents CanRisk, a hierarchical fuzzy rule-based model for the seismic risk assessment of URM/RC buildings;
- Chapter 4: Presents a sensitivity analysis and uses data collected from the 2011 Christchurch Earthquake as verification of the CanRisk model;
- Chapter 5: Introduces Urban Rapid Assessment Tool (Urban RAT), a tool for the collection of building data in dense urban areas, and presents the results from the application of the tool to the City of Ottawa;
- Chapter 6: Presents the regional seismic risk assessment of URM and RC buildings in the downtown core of the City of Ottawa and a case study of detailed seismic risk assessments of a handful of URM buildings;
- Chapter 7: Summarizes concluding remarks and provides some recommendations for future research efforts.

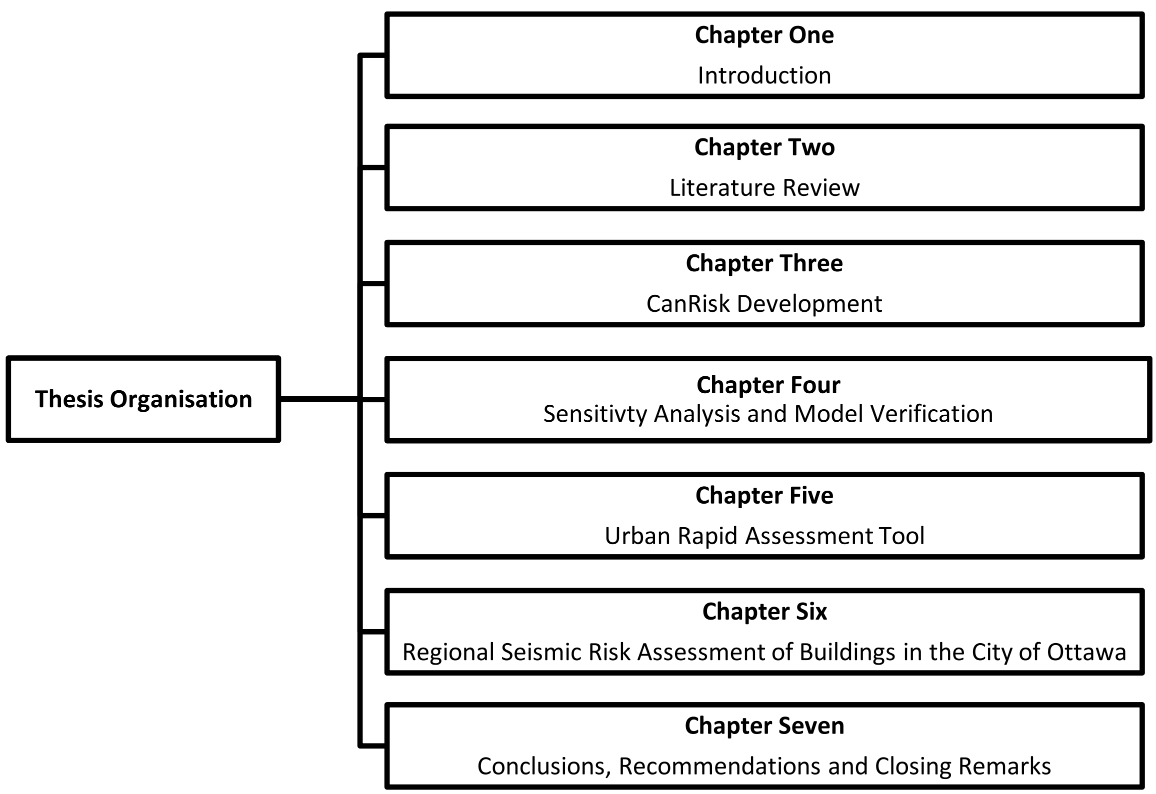


Figure 1-2: Thesis Organization

Chapter 2
Seismic Performance of URM Buildings and Review of Seismic Risk Assessment Procedures and Literature

2.1 Introduction

This chapter presents a review of available seismic risk assessment procedures and tools for building structures, as well as a review of previous literature on the topic. After defining seismic risk, the importance of data collection using GIS-based tools is discussed. Various methodologies for assessing seismic risk are then summarized and discussed, including the tools specifically developed for URM buildings. A review of fuzzy set theory and fuzzy rule based modelling in handling the subjectivity and uncertainty involved in risk evaluation process is presented, as these analytical tools are used in the current project.

Finally the literature review includes a detailed state-of-the-art review of the seismic vulnerability of URM buildings. This section defines URM building characteristics, reviews code development related to URM, discusses known seismic deficiencies in URM and summarizes past performance of URM buildings in documented earthquakes.

2.2 Seismic Risk

The seismic damageability of a building is defined as the probability (or likelihood) of loss or damage resulting from a given level of earthquake for a given building. The measure of damageability depends on several factors, such as the unique attributes of a building being evaluated and the intensity of a given earthquake. This information can be used to assess seismic risk as the likelihood of the impact of building damageability in terms of expected economic and human loss, such as the extent of property damage and the potential number of casualties following an earthquake (Tesfamariam, 2008; Coburn and Spence, 2002).

Seismic risk assessments require consideration of site seismic hazard, building vulnerability and building importance/exposure. Site seismic hazard is related to seismicity of a given region and building site conditions. Building vulnerability takes into account the inherent building characteristics and unique attributes that can influence seismic performance of a structure. Building importance reflects the relative importance of a building to a community by identifying building class and human/economic impact.

2.3 Building Data Collection and GIS

Proper disaster planning, mitigation techniques, preparedness and response procedures can all be handled through a Geographic Information Systems (GIS). The seismic risk assessment of densely populated urban areas requires intense collection of building data. GIS tools can facilitate rapid data entry, analysis and visualization of spatial data. These tools have been utilized in many emergency management applications (Herold and Sawada, 2012) and they provide an efficient toolset for mapping the effect of hazard across a region for loss estimation studies (Tari and Tari, 2002; Coburn and Spence, 2002). As the consequences of an earthquake vary spatially, GIS-based tools link the event of an earthquake with hazard specific information such as surficial geology (Motazedian et al., 2011) and structural variations. Success in mapping the spatial variability in seismic risk outcomes requires a well-developed database of building structures. A building database can be effectively populated directly within a GIS system.

A well-developed building inventory hinges on the information that is collected. Detailed data collection forms and guidelines exist for loss estimations as outlined in FEMA 154 (ATC, 2002) and HAZUS®MH (ABS Consulting and ImageCat, 2006). They include parameters such as construction type and year of construction which are good indicators of expected seismic performance. These parameters can be collected using GIS-based tools and used to populate inventory databases used for earthquake loss estimations.

2.4 Seismic Risk Assessment

2.4.1 Existing Methodologies for Assessing Seismic Risk

Various methods have been proposed in the literature to carryout seismic risk loss estimations, including: empirical/statistical models, heuristic models and analytical/mechanistic/theoretical models (Tesfamariam, 2008). The following section provides a review of existing methodologies proposed in various countries for the seismic risk assessment of buildings.

2.4.1.1 Japan

Otani (2000) developed a simple screening procedure for the seismic vulnerability evaluation of existing reinforced concrete buildings in Japan. The need for evaluating existing building stock was recognized after a review of damage statistics from major earthquakes, such as the 1995 Hyogo-ken Nanbu earthquake in Kobe city. To evaluate existing RC buildings various parameters were identified, including: strength and deformation capability of members, material properties, structural configuration, site condition, quality of workmanship, importance of buildings, year of construction and effect of non-structural elements. The evaluation procedure is derived from building design equations and results in various levels of screening. The lateral strength and deformation capacity is first estimated on the basis of actual dimensions and material properties of building elements and records from previous earthquakes in Japan through indices. Thereafter, if the building has been identified as deficient in strength and/or capacity then the building is evaluated using a nonlinear procedure.

2.4.1.2 USA

Hazards U.S. Multi-Hazard (HAZUS-MH) is a wide-ranging GIS-based loss estimation tool developed by the Federal Emergency Management Agency (FEMA) and National Institute of Building Sciences (NIBS). Currently, the U.S. version (see FEMA, 2011) has the ability to estimate effects of earthquake, flood and hurricane winds while the Canadian version includes earthquake and flood estimations (see Ploeger et al., 2010). HAZUS-MH is very comprehensive in terms of earthquake loss estimations. It has the ability to estimate physical losses such as damage to buildings and debris, social losses such as casualties and shelter requirements and economical losses including the replacement costs of buildings. Since it is extended into a GIS platform it allows the user to map the losses, so it can be easily visualized to various stakeholders. It is a useful tool for disaster management purposes as it can help estimate losses and better plan, prepare for and anticipate the nature and scope of a future earthquake. It is noted that a well-developed building inventory is required as input into the HAZUS-MH software; otherwise default settings are used, which can introduce significant error within the evaluation. HAZUS uses various methodologies to determine loss. For instance, within the earthquake loss estimation module, the evaluation of building damageability is determined through fragility and capacity curves; in the case of casualty estimations it uses a modified version of ATC-13. There are other methodologies that exist within the HAZUS-MH program for various evaluation parameters. The results of the evaluation process are presented by aggregated areas and include a summary of the inventory and losses over a census space. However, it is also possible to conduct a building specific evaluation (such as an individual building of interest).

The FEMA 154 report (ATC, 2002), provides a rapid visual screening (RVS) procedure for seismically vulnerable buildings. The approach consists of a sidewalk survey for a given building using a data collection form based on a visual assessment. The information collected using the data form includes: identifying the building type (Seismic Force Resisting System), year of construction, number of stories, building area, building use, building occupancy, structural irregularities, soil type and falling hazards of non-structural components. Once the input fields have been identified on the data form, a basic structural hazard score is given that relates to the probability of building collapse under severe ground shaking. A greater score from the evaluation of a building results in a greater expected seismic performance. From this evaluation process, buildings identified as seismically deficient are recommended for detailed evaluation. Often the FEMA 154 RVS scores of buildings are compared with the results from HAZUS-MH. The FEMA 154 report is typically used as an inexpensive, rapid screening process on a regional scale to determine the expected seismic performance levels of a community's building stock.

The handbook for the seismic evaluation of buildings, FEMA report 310 (ASCE, 1998), is considered an advanced seismic evaluation procedure with a three-tier evaluation procedure with increasing complexity of analysis at every level. The handbook identifies evaluation requirements using basic building information such as building type, level of seismicity, and testing procedures from information/material gathered on-site to establish level of performance. Tier 1 represents the initial screening phase, and is conducted using checklists. At this stage, buildings are grouped as compliant or non-compliant with the current provisions of the FEMA 310 handbook. The buildings containing deficiencies or not meeting the requirements of FEMA 310 proceed to Tier 2 evaluation. In the Tier 2 phase, buildings are analyzed using common linear static or dynamic analysis methods to identify the effect of deficiencies found in Tier 1. Buildings that are considered unsatisfactory based on Tier 2 requirements proceed to Tier 3 evaluation. In the Tier 3 evaluation, a detailed nonlinear analysis is conducted in order to identify potential mitigation actions to bring the building to an acceptable level of seismic performance.

2.4.1.3 New Zealand

A document by the New Zealand Society for Earthquake Engineering, (NZSEE, 2006), provides a step-by-step evaluation of the seismic performance of buildings with different marterial types and configurations. The procedure begins with an initial evaluation procedure (IEP) based on a visual screening process and a structural score is used as an indicator of potential building damage. Two scoring factors are obtained, the first is a basic score, which reflects the standard for the orginal design and the poential earthquake hazard of the building in question, the second is a modication to the basic score to account for any seismic deficiences found in the building. Moreover, building importance is established by building area, occupant density and potential casulasties, and is combined with structural score to determine if a detialed assessment is required. If a building is evaluated using the detailed structural assessment, the building is investigated at the component level. Finally, force-based and displaced-based methods are utilized for the detailed assessment of later force-resisting elements of the building in order to determine the overall potential risk and estimated structural damage.

2.4.1.4 Europe – Eurocode 8 – part 3

Part 3 of Eurocode 8 (CEN, 2005) provides guidelines for the seismic assessment and retrofit of existing damaged or undamaged structures, and has been adopted by a number of European countries. The evaluation process considers the seismic vulnerability of buildings for three levels of performance (near-collapse, significant damage and damage limitation) based on three levels of return period (2475 year, 475 year and 225 year return periods). The structural performance is evaluated by linear or non-linear analysis where a displacement-based approach assesses the performance of buildings. Information relevant to the building's geometry design details and materials are obtained for the structural assessment. Following the analysis, verification at the component level is conducted depending on the type of failure: ductile or brittle. Ductile elements are assessed by limiting response to a permissible deformation, while brittle elements are assessed by limiting response to a maximum strength or force. Lastly a confidence factor is introduced to handle any uncertainty in the evaluation. It is noted that this modification to the evaluation of the performance of a structure varies based on the type of analysis carried out by the evaluator (linear or non-linear analysis). In addition, the document provides details for structural retrofitting and decision-making.

2.4.1.5 Canada

The SCREEN tool (Saatcioglu et al. 2011) presents a rapid seismic screening procedure. The tool was developed by updating the earlier seismic screening manual originally developed by the National Research Council (NRC) of Canada in 1992 based on the FEMA 154 rapid visual screening process (ATC, 2002). The tool was developed to provide seismic screening of buildings based on field inspections or analysis of building drawings. It is used to identify buildings that require further investigation to assess seismic vulnerabilities and allow for a prioritization of seismic retrofit needs. The method used in the Screening Manual is based on the computation of a seismic priority index (SPI) as the product of factors that affect building performance during earthquakes. The factors taken into consideration include: seismicity, soil condition, type of structure, building irregularity, building importance and non-structural hazard (further detailed in Table 2-1). Each independent factor is computed empirically or by observations using guidelines in the screening manual. The higher the SPI value, the higher the vulnerability of the building being evaluated. The buildings are then ranked by their scores, where the scores are divided into four different categories of priority (low, medium, higher and potentially hazardous). The tool allows for application in any city in Canada for which the uniform hazard spectrum (UHS) is defined in the National Building Code of Canada (NBCC 2005) and provides a tool and rapid visual screening procedure for identifying potentially seismically deficient buildings in Canada.

CanRisk, a loss estimation program for the seismic vulnerability assessment of reinforced concrete buildings is a tool developed by Tesfamariam and Saatcioglu (2008). The tool integrates site specific spatial information such as NBCC-based soil conditions and ground motions with detailed user-input building-specific data using hierarchical fuzzy-ruled based theory (see Figure 2-1). The program is modular in that it evaluates different parameters to establish seismic risk of buildings. Specifically, CanRisk's risk-based assessment integrates site seismic hazard, building vulnerability (likelihood of failure) and building importance/exposure factor (consequence of failure) as illustrated in Figure 2-2. Currently, the program allows for the evaluation of reinforced concrete buildings; and part of the scope of this thesis is to expand the program's functionality to allow for the evaluation of unreinforced masonry (URM) structures. The program output establishes structural building damage levels and a risk index for a given building. More details related to fuzzy logic and how it relates to the CanRisk program can be found in Chapter 3.

Table 2-1: Seismic Screening Manual Factors Affecting Building Performance

Factor	Description
A – Seismicity	Utilizes the year of construction to compare the seismic force design levels between NBCC-1990 and NBCC-2005, reflecting relationship between seismic hazards of 1985 and 2005. The relationship with earlier seismicity can also be established by using multipliers given in the Screening Manual.
B – Soil Condition	Equates NBCC-2005 new soil classifications and related period dependant acceleration-based and velocity-based site coefficients with soil conditions specified in earlier codes to establish the correlation of amplification /de-amplification of soil conditions.
C – Type of Structure	Reflects the building's toughness. Compares NBCC-2005's more refined approach for categorizing different building types (R_dR_o) with earlier editions of the NBCC (coefficient K and then by coefficient R) which reflects the building's ability to deform in the inelastic range.
D – Building Irregularity	Captures any structural irregularity within a building (vertical irregularity, plan irregularity, pounding, etc…) known to produce seismic deficiencies.
E – Building Importance	Occupancy related importance factor which incorporates occupied area, occupancy density and a duration factor. Identifies critical infrastructure, post-disaster buildings and high-occupancy buildings.
F – Non-Structural Hazards	Highlights any hazard related to non-structural elements. Considers objects that are falling hazards to life and damage to vital operations of strategic, post-disaster facilities.

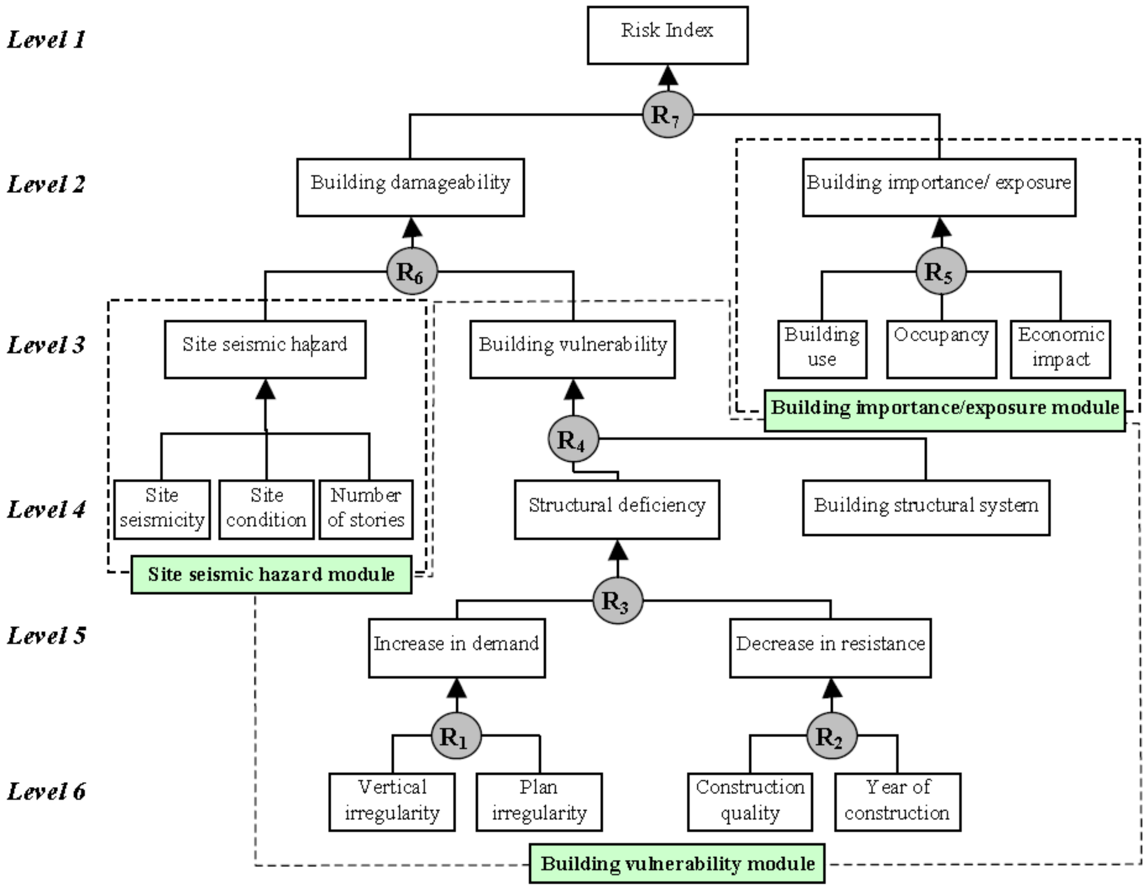


Figure 2-1: Hierarchical seismic risk analysis of RC buildings

(Tesfamariam, 2008)

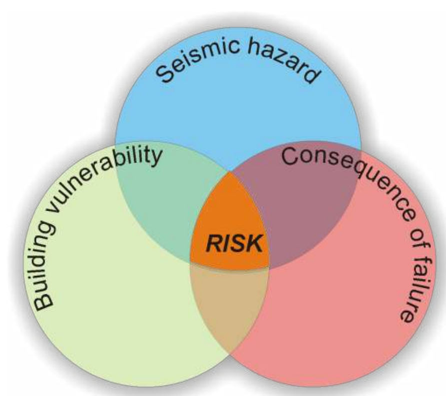


Figure 2-2: CanRisk Earthquake Risk Assessment Modules

(Tesfamariam, 2008)

2.4.2 Existing Methodologies for Seismic Risk Assessment of URM Buildings

In addition to the general seismic risk assessment methods described in Section 2.4.1, researchers have also proposed methodologies specifically for the seismic risk assessment of URM buildings. The following sections provide reviews of some of the methods proposed in the literature.

2.4.2.1 Turkey

Erberik (2010) presented an engineering application for the seismic vulnerability assessment of URM buildings in Istanbul, Turkey, which is known to be an urban city prone to moderate to severe earthquakes. In the investigation presented by Erberik (2010), a building inventory of approximately 20,000 masonry buildings was examined in a two stage process. The first stage included a preliminary sidewalk survey of unreinforced masonry buildings, while the second stage followed a more detailed procedure of those URM buildings tagged as highly vulnerable during the first stage evaluation. The author noted that common seismic deficiencies found in URM buildings include: poor masonry material quality and characteristics, plan geometry/irregularity, irregular wall openings, the number of stories and insufficient wall lengths, which are in turn used as key parameters for structural evaluation during the sidewalk survey. Results from the sidewalk survey indicated that most of the URM buildings did not conform to the Turkish Earthquake Code regulations. The risk assessment in the first stage involved both an identification of seismic hazard as well as the assessment of building vulnerability. Seismic hazard was determined by using peak ground accelerations (PGA) with a probability of exceedance of 10% in 50 years. Building vulnerability was established by using fragility curves for 120 different classes of masonry buildings, where each class contained the different structural deficiencies/parameters previously mentioned. Subsequently, a performance score was assigned based on damage state probabilities with corresponding damage state multipliers for the assigned PGA value and were ranked accordingly. In addition to the performance score, the buildings were also given a level of low, medium or high risk. A threshold was established and the masonry buildings below the limit were required for a detailed inspection. As a result of the performance score, based on established probability levels by the author, 4,105 of 19,189 buildings were considered to pose significant risks. In the second stage, a more in-depth investigation of the buildings identified in stage one was conducted. Detailed information including structural layouts and geometrical properties were obtained for the second stage. Based on empirical equations, structural parameters were computed such as: lateral shear resistance of wall segments and plan eccentricities which were compared with current Turkish Earthquake Code regulations. The results from stage two concluded that 2,786 buildings are classified as high risk. Overall the paper captured the inevitable vulnerability of masonry buildings in Istanbul, Turkey and proposed a feasible pre-earthquake risk assessment procedure for disaster management and risk mitigation purposes.

2.4.2.2 New Zealand

In New Zealand, an investigation on the seismic vulnerability of existing URM building stock was conducted. Russell and Ingham (2010) applied a methodology to group URM buildings into various typologies for the purposes of seismic assessment based on the knowledge that building configuration plays as one of the most important roles. The key features for differently defined typologies included building storey height and building footprint. The main typology types are listed in Table 2-2 and can be further divided into subclasses based on plan configurations, such as wall distributions and arrangements. The typology class of URM buildings were also identified with an expected importance level according to the New Zealand code regulations.

Table 2-2: Classification of New Zealand URM Building Typology

Type	Description
A	One storey, isolated
B	One storey, row
C	Two storey, isolated
D	Two storey, row
E	Three + storey, isolated
F	Three+ storey, row
G	Institutional, religious, industrial, other

Based on the data obtained from city organizations, old historical data as well as other census tracts, an estimated amount of 3,750 URM buildings existed in 2010. In terms of populating the database, two different approaches were used. The first approach involved aggregating and identifying the location of URM buildings and their distribution between the provinces of New Zealand, while the other approach gathered similar information but also provided an estimated financial value to the existing building stock. Both methods assisted in identifying the location, age of construction, building height and financial value for the URM building stock. Results obtained show most of the URM in New Zealand are one storey buildings built between the years 1920 and 1930.

For evaluation purposes, the expected vulnerability of URM buildings was established using the guidelines outlined by the New Zealand Society for Earthquake Engineering (NZSEE). The initial evaluation procedure (IEP) provided a screening method to determine a building's performance in the event of an earthquake. The assessment involved comparing the expected building performance with current New Zealand building practices to determine a "Percentage New Building Standard," (%NBS). This score value captures the building's structural weaknesses, seismicity potential, structural irregularities and potential for pounding. According to the methodology, a %NBS score of 33 or less indicated that the building was a significant earthquake risk. A score between 33 and 67 identified the building as being a potential earthquake risk and finally a score of greater than 67 indicated that the building was unlikely to be a significant risk. The %NBS was calculated for the URM building stock using the results from the initial screening evaluation. As a result, 35% of the total inventory scored less than 33 %NBS and 52% scored between 33 and 67 %NBS indicating that 87% of the total URM building stock in New Zealand as being seismically vulnerable. This research effort indicates that significant attention must be given to URM buildings and indicates the need to identify vulnerable URM buildings.

2.4.2.3 India

Arya (2008) presented a two-step procedure to assess the seismic vulnerability of URM buildings in India. Data obtained from a census survey of 2001 identified 84.7% of 240 million housing units were constructed of masonry. Lessons learned from previous earthquakes in India (1993 Latur, 2001 Kachchh), demonstrated that masonry buildings performed the poorest. The first step to the assessment included a rapid visual screening (RVS) procedure. Within this step, the evaluator identifies the main seismic force-resisting system (SFRS) and identifies any potential performance modifiers to the seismic performance of the building (structural and non-structural) using a fill-in form. Assessment parameters include: building type such as type of masonry unit (stone, brick) and the type of mortar (mud, lime, cement), number of stories, year built, area, occupancy, soil type, foundation type, floor type, building irregularities and falling hazards (chimneys, parapets, cladding). Hazard is determined using code-based seismic intensity, and the damageability grade of the building is defined using the European macro-intensity scales. In the second step, a thorough assessment is conducted to identify if a building is non-compliant to code regulations in India. If deficient, the building is recommended to be retrofitted or strengthened as necessary. Details investigated in the second step include an investigation of the structural drawings in order to determine height/openings of walls, reinforcement (if any), unrestrained non-structural components and diaphragm types. Occasionally, the evaluator will conduct material tests from the building to provide further information for an in-depth investigation. Overall the paper outlines a two-step procedure to assess masonry buildings in India in order to identify seismically deficient masonry buildings requiring retrofit in order to minimize damage and loss.

2.4.3 Uncertainty in Seismic Risk Assessment and Fuzzy Logic as Solution

It is important to note that the evaluation of seismic risk of a building is complex and involves uncertainty. As outlined in the previous sections, various methods have been proposed in the literature to carryout seismic risk loss estimations. Recently, researchers have also proposed the use of fuzzy set theory and fuzzy rule based modelling to handle the subjectivity and uncertainty in the evaluation process (Tesfamariam, 2008).

The use of fuzzy logic within engineering applications has become a popular method to define and quantify uncertainty in engineering systems that arise from imprecision, ambiguity, and lack of data or knowledge. Fuzzy logic is a method that allows for approximate values and inferences as well as incomplete or ambiguous data sets as opposed to only replying on crisp data which involves the traditional binary yes/no or true/false choices (Tesfamariam, 2008). The limitations of human linguistics, such as the interpretation of descriptors (high, hot, warm, low), demonstrate the uncertainty or inexactness of meaning in language as interpretations can vary from individual to individual. For example, while conducting a sidewalk survey of a building, an engineer may assess the vertical irregularity of a building differently than another based on individual’s familiarity, experience and knowledge. In the assessment of seismic risk, the use of fuzzy logic can handle the inevitable uncertainty embedded within the evaluation process by assigning different membership values (or values of truth) by defining different membership functions as seen in Figure 2-3. Further discussion on fuzzy logic as it applies to CanRisk will be presented in Chapter 3.

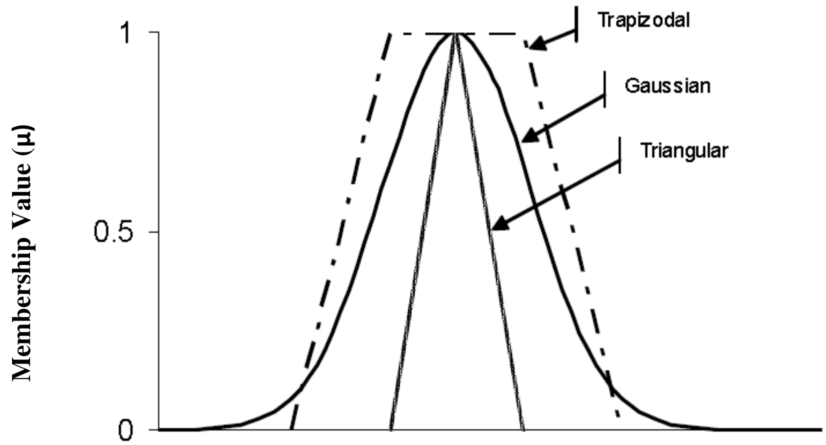


Figure 2-3: Typical Fuzzy Membership Functions

(Tesfamariam, 2008)

2.5 Seismic Vulnerability of URM - State-of-the-Art Review

Unreinforced masonry (URM) buildings have consistently performed poorly in earthquakes (Bruneau, 1994). URM building construction is known to be the most seismically vulnerable construction type. Many densely populated cities in Western and Eastern Canada (Vancouver, Montreal and Toronto) have large inventories of unreinforced or poorly designed masonry structures (Hodgson, 1945; Bruneau and Lamontgne 1994). Although measures have been taken to rehabilitate and increase the seismic resistance of important and historic structures, many existing unreinforced masonry structures have not been retrofitted and remain at risk in the event of a large magnitude earthquake (Bruneau, 1994; Bruneau and Lamontagne, 1994). This section defines URM building characteristics, reviews code developments related to URM, reviews known seismic deficiencies related to URM, and summarizes past performance of URM in documented earthquakes.

2.5.1 URM Building Type

Unreinforced masonry is considered to be the oldest building material construction type (Hess, 2008). The majority of the URM building stock in North America was built before

1940, prior to the development of modern seismic design criteria. This construction type was permitted in areas of moderate or high seismicity until the early-1950s in the United States and the mid-1970s in Canada (Brzev, 2010).

Unreinforced masonry buildings consist of bricks/blocks bonded together by mortar that make up the structure. URM buildings are "unreinforced" with the implication that reinforcing steel bars are not embedded within the cells of the units in comparison to more modern reinforced masonry buildings. The most common unit type is solid clay-brick, but other types of masonry units exist (concrete block and adobe). These buildings are commonly used for commercial, residential or industrial purposes, built with no greater than six stories. URM buildings are usually load-bearing wall structures (see Figure 2-4) that transfer gravity loads to the foundation of the building (ATC, 2002).

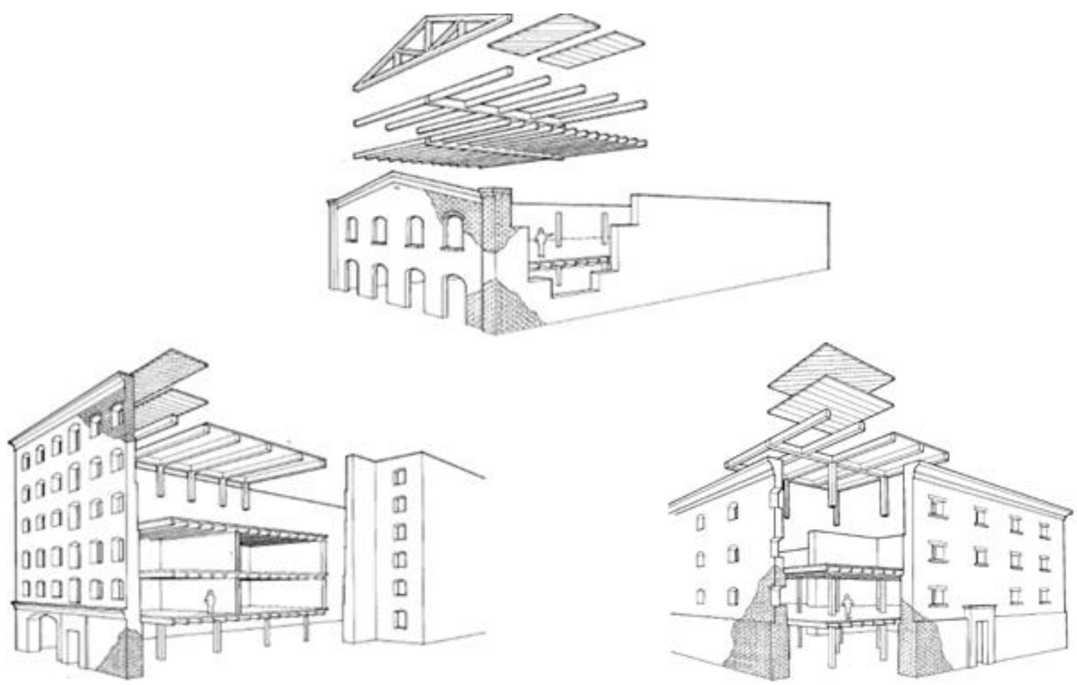


Figure 2-4: Typical construction of URM bearing-wall buildings in North America

(ATC, 2002)

2.5.2 Structural Components

2.5.2.1 Walls

The main structural component of a URM building is the load-bearing wall. There are four different main sub-components that can characterize variations in load-bearing wall configuration, including: solid walls, piers, spandrels and gables (Figure 2-5) (ATC, 1998). URM buildings comprised of solid wall components, as seen in Figure 2-5a, are constructed with very few or no window or door openings. While URM buildings containing windows and door openings (Figure 2-5b) are constructed with structural masonry piers and spandrels where masonry piers are oriented vertically and spandrels oriented horizontally (both typically being bounded by the window or doors in a URM structure). Finally, the gable component of a load-bearing URM sits at the top of a wall of buildings having a pitched-roof (see Figure 2-5c).

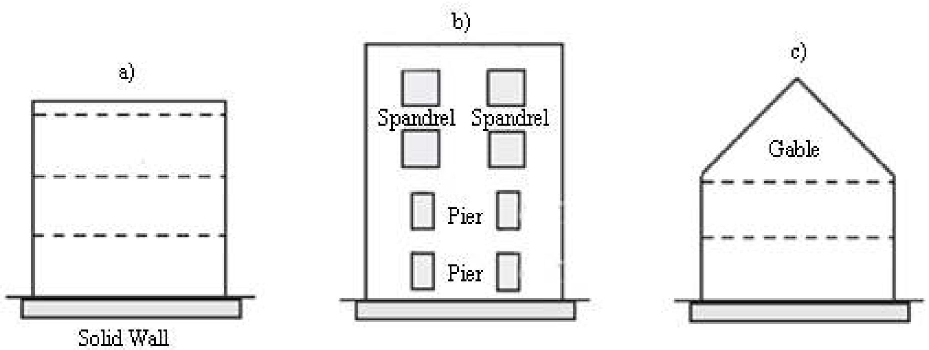


Figure 2-5: Wall components of URM: a) Solid Wall, b) Pier and Spandrel, and c) Gable

(ATC, 1998)

2.5.2.2 Diaphragms

The structural floor and roof diaphragms are the horizontal floor or roof elements that form part of the lateral force-resisting system (LFRS) of URM buildings. Under seismic loads, the basic function of diaphragms is to collect inertia loads arising from mass and to distribute the lateral loads to vertical elements in the LFRS. In terms of seismic behaviour, diaphragms can be classified as rigid, semi-rigid or flexible (ATC, 1998). In the case of rigid diaphragms, distribution of horizontal forces to vertical elements is in proportion to their relative stiffness and lateral deformations are significant in the diaphragm when compared to the vertical elements. In the case of flexible diaphragms, this distribution of forces is independent of the relative stiffness of vertical elements and lateral deformations are significant (Anderson and Brzev, 2009). In the case of masonry structures most structures with concrete roofs and floors can be classified as being rigid while wood or metal roofs and floors can be classified as flexible. Wall-Diaphragm Ties are components that connect masonry walls to floor or roof diaphragms in order to secure separate building components together.

2.5.2.3 Non-structural Components

Non-structural components in URM buildings include elements such as parapets, chimneys, appendages and any ornamentation (ATC, 2011). Parapets are wall-like barriers at the edge of a roof, where these short extensions of walls above the structure typically occur at the perimeter of the buildings and are primarily present for fire safety or architectural purposes as illustrated in Figure 2-6. The chimneys are components that project above the roof of the building and when subject to seismic actions, they act as cantilevers which rock on their supports at the roof line. If sufficiently accelerated by the earthquake, they will topple over (Ingham and Griffith, 2011a). Appendages include: veneers, cornices corbel and any non-structural component that is likely to create a falling-hazard.

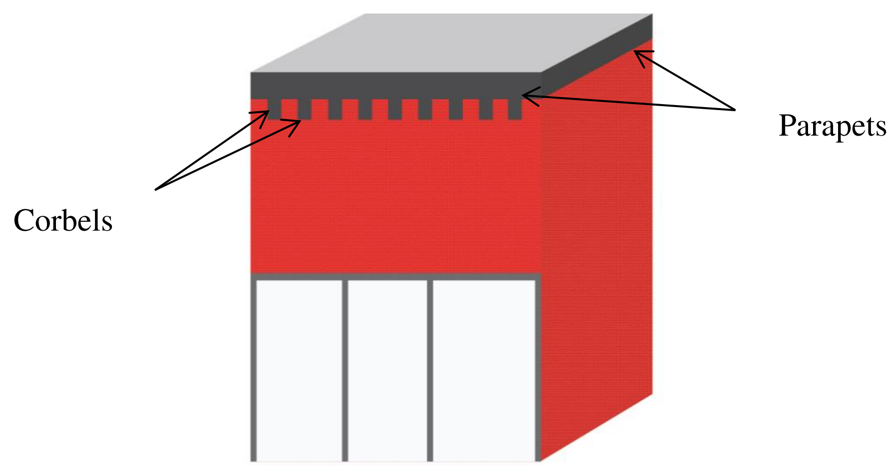


Figure 2-6: Non-structural Components of URM Buildings

2.5.3 Methods for Identifying URM construction

The following suggestions are intended to assist in identifying if a building is constructed of URM (ATC, 2002):

- If the building contains URM load-bearing walls; no columns and/or many exterior walls present in the building may suggest that the building is of URM construction.

- The year of construction can provide an indication to determine the age of a building. Age provides details to whether URM construction was permitted during a specific time of a building code (if present) with seismic provisions.
- Architectural features such as arch window heads, parapets, corbels, etc…
- The unit brick pattern of the building. Header bricks in the wall surface (Figure 2-7) are typically found in URM structures
- Anchor plates indicating retrofit of a URM building (wall-to-floor/roof diaphragm ties as seen in Figure 2-8).
- The type of mortar utilized in bonding the units. Lime mortar was used in older URM buildings, known to cause poor earthquake performance. A simple test can be conducted by scratching the mortar with a coin to determine if the mortar is soft.

For more detail refer to ATC, 2002.

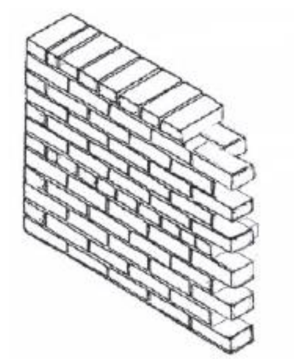


Figure 2-7: Header bricks, indiciating no cavity for rebar to be placed

(ATC, 2009)

Figure 2-8: Example of through ties connecting the roof diaphragm to load-bearing wall

(Ingham and Griffith, 2011a)

2.5.4 Code Development and Design Provisions

2.5.4.1 United States and New Zealand Codes and Standards

Although the building code and masonry design provisions vary by region within the US, the current International Building Code (IBC), which is adopted throughout most of the US, effectively bans all URM construction in areas where moderate to strong earthquakes are expected. Figure 2-9 illustrates the areas within the US where URM construction is currently not permitted. Although the IBC recognizes these areas, existing URM buildings can still be found in areas of high seismicity built before seismic design requirements such as California. Lessons learned from previous earthquakes in the US such as the 1933 Long Beach earthquake, where many URM buildings were severely damaged, increased the awareness of the hazard that URM buildings pose. Many retrofit ordinances and were implemented after the Long Beach earthquake in Los Angeles to URM buildings, but many other cities did not follow by example (ATC, 2009; Klingner, 2004). Performance of retrofitted URM buildings in recent earthquakes, such as the 1994 Northridge earthquake, provides evidence of the success of risk reduction and retrofit programs implemented in the United States to bring URM buildings to a satisfactory level of seismic performance.

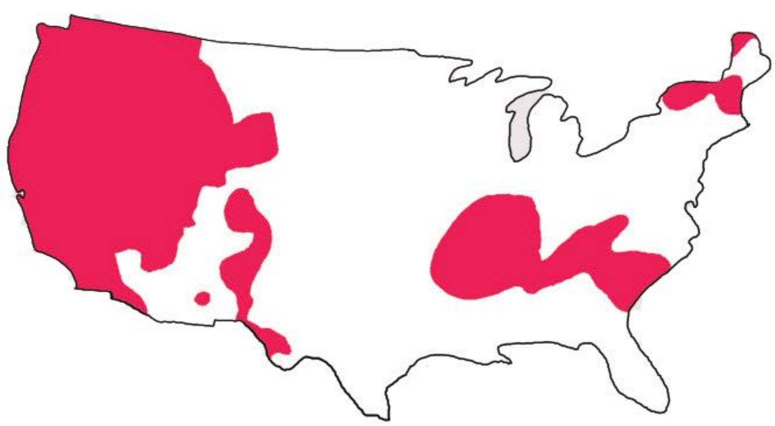


Figure 2-9: Areas where current building code regulations prohibit URM construction in the U.S.

(ATC, 2009)

Similarly, in New Zealand, the first standardized masonry guidelines were implemented in 1948 following the 1931 Napier earthquake that killed approximately 260 people. In 1968, legislation recognized the potentially poor earthquake performance of the existing URM building stock. URM buildings built with insufficient capacity to resist earthquake design forces of less than 50% outlined by the code for that period were defined as buildings at risk. Many URM buildings were required to be demolished or retrofitted in order to ensure a proper level of safety. The most recent code development in New Zealand is defined in the Building Act of 2004, which outlines retrofitting standards to earthquake prone buildings in order to ensure a building is improved to an appropriate seismic standard otherwise the building should be demolished (Dizhur et al., 2010; Smith and Devine, 2011; Russell et al., 2006). Despite the above steps, the hazards associated with the URM buildings stock was exposed during recent earthquakes in the region (e.g. 2011Christchurch Earthquake)

2.5.4.2 Canadian Codes and Standards

Most unreinforced masonry (URM) structures in Canada were built before the implementation of stringent earthquake design requirements. It is important to note that unlike California, which banned the construction of unreinforced masonry structures in 1933, Canadian building codes permitted URM buildings to be constructed regardless of location or seismic zone until mid-1970s (NRCC, 1975; ATC, 2009). The 1975 edition of the National Building Code of Canada, prohibited the construction of URM buildings in moderate to severe seismic regions (where roughly 40% of the Canadian population lives), and required reinforced masonry to be a mandatory type of construction in masonry type structures (Statistics Canada, 2011, Bruneau, 1995; Brzev, 2010). Based on this basic information, masonry structures built prior to 1975 should be flagged and evaluated in terms of seismic vulnerability. Table 2-3 summarizes the provisions of the CSA standard for masonry design and development of NBCC seismic provisions requiring reinforced masonry (RM) construction in areas of moderate to high seismic zones in Canada. (CSA, 1978; NRCC, 1975)

Table 2-3: Masonry Code Development in Canada (Brzev, 2010)

Year	Development
Pre-1975	• No seismic provisions related to URM or RM construction
1975	National Building Code of Canada: • Use of reinforced masonry mandatory in seismic zones 2 and higher ($Za \geq 2$ or $Zv \geq 2$) • Latest edition in 2010
1978	CSA S304.1-78 Masonry Design for Buildings : • First edition of the Canadian Masonry Code (followed by 1984, 1994, 2004)

2.5.5 Overview of Known Seismic Deficiencies

2.5.5.1 Structural layout

The structural configuration of a URM building plays an important role in determining its seismic performance. Most often, URM buildings with the same story height and rectangular, box-like configurations perform much better than URM buildings with irregularities such as soft/weak storeys, asymmetrical window openings, few partitions and cross-walls unevenly distributed and oriented in one principal direction (Hess, 2008).

2.5.5.1.1 Building Typology

Although typology is not directly a seismic deficiency, results from post-earthquake assessments of existing URM structures demonstrate the relationship between the level of damage and a building's typology. As seen in Figure 2-10a, constructing multiple URM buildings in a row provides additional resistance to ground motion when compared to the performance of URM stand-alone buildings. This is likely due to the presence of additional load-bearing walls behaving as shear walls between buildings. In addition, Figure 2-10b shows that URM buildings positioned in the middle of a row are considered to be somewhat protected by the end-of-row URM buildings (known as "bookend behaviour") (Ingham and Griffith, 2011b).

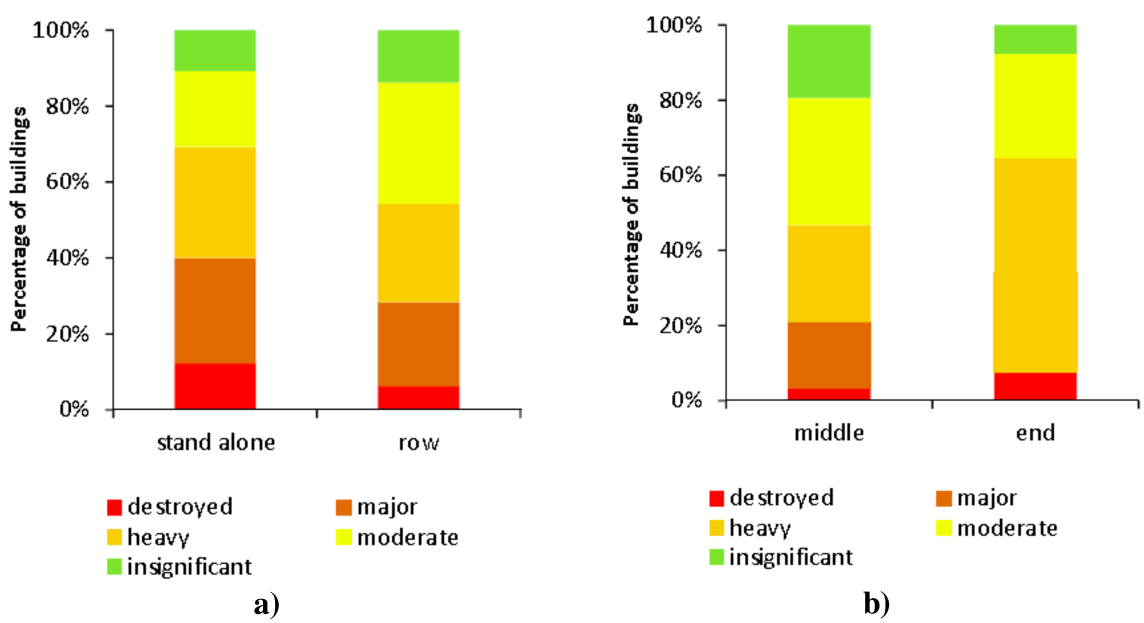


Figure 2-10: ATC-13 Damage Classification for: a) stand-alone and row URM buildings, b) position in row URM buildings

(Ingham and Griffith, 2011b)

2.5.5.1.2 Soft / weak Story

In order to increase the openness at the ground level of a URM building, often the structural elements are not continued to the foundation creating a soft/weak story in the building (ATC, 1998; Bruneau, 1994a). This type of irregularity is most commonly found in URM buildings operating as retail stores that contain open street façade fronts at the ground level parallel to the direction of the street as illustrated in Figure 2-11. Since the concentration of loads is greatest at the base of a building, a soft or weak story between the first and second floor results in a serious deficiency. A weak story results in inadequate strength when compared to stories above and can often result in failure of that story. In a soft story, the level in question has reduced stiffness or more flexibility when compared to the story above. A soft story can result if the first floor height is significantly taller than those above causing a large discrepancy in stiffness. Similarly, an open first floor that supports heavy structural or non-structural walls above can represent a special case of a soft and weak story problem (EERI, 2006).

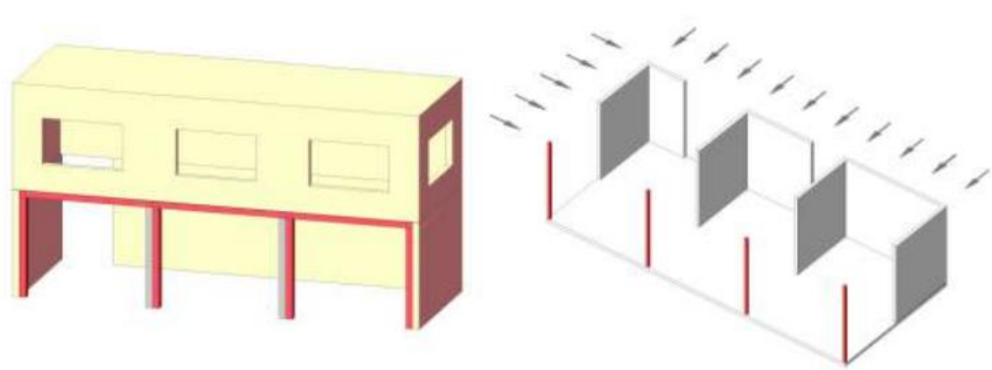


Figure 2-11: Typical Soft Story in a URM building

(Castro & Associates, 2007)

2.5.5.1.3 Wall opening configuration

Openings of different sizes, openings misaligned in the horizontal and vertical direction and a variable number of openings per story (see Figure 2-12) are all factors that can influence the seismic performance of URM building structures. These irregularities introduce a non-uniform distribution of gravity loads and an unfavourable concentration of seismic load in parts of the wall, increasing the structure's seismic vulnerability (Parisi and Augenti, 2012).

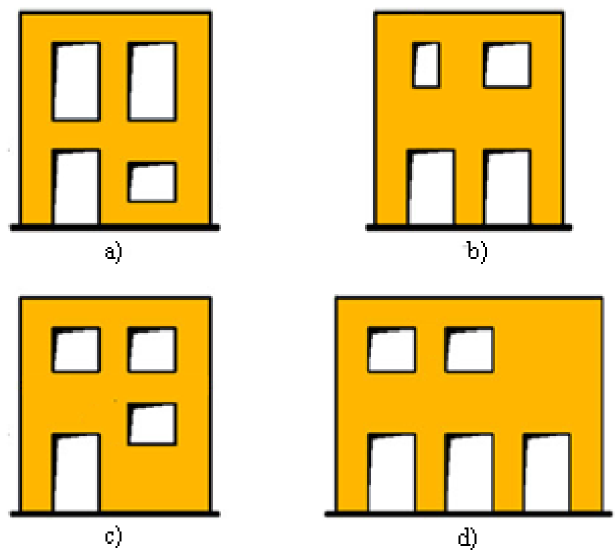
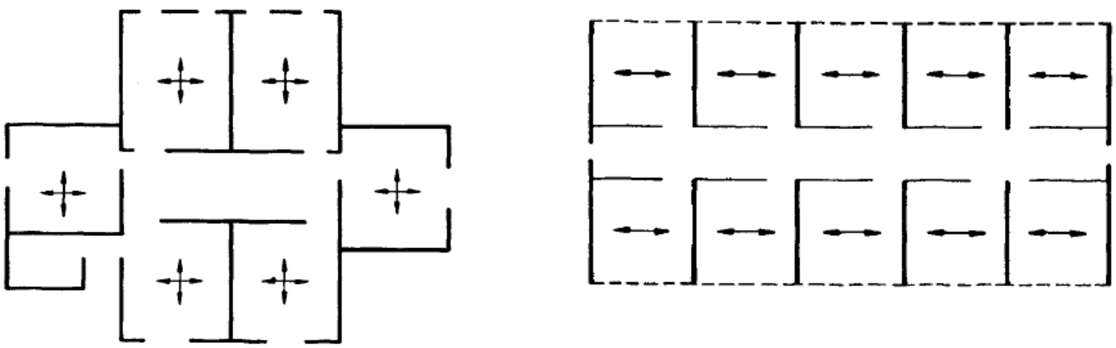


Figure 2-12: Irregular wall opening configurations with: a) horizontal irregularity, b) vertical irregularity, c) offset irregularity, and d) variable openings per floor irregularity

(Parisi and Augenti, 2012)

2.5.5.1.4 Torsional irregularity

Torsional effects result in a lack of balance between the elements of the SFRS and the overall arrangement of the building mass. The location of walls built to form the overall structure influence the eccentricity between the center of mass and the center of resistance. A large eccentricity can result in undesirable concentrations of stress and torsion causing twisting action in the building. A reasonable balance between the distributions of walls in the principal directions of the building is essential in order to provide adequate resistance to lateral forces in both directions as illustrated in Figure 2-13a. URM buildings with square or rectangular configurations with well distributed walls will generally have a greater torsional resistance than buildings with less evenly distributed lateral force resisting walls or buildings with walls only distributed in one principal direction as seen in Figure 2-13b (Ingham and Griffith, 2011a; Hendry et al., 1997). In addition, URM buildings located at the corner of an intersecting street often with asymmetrical wall arrangements and plan configurations may exhibit a large torsional response due to a large eccentricity.



a) b)

Figure 2-13: Typical wall arrangements in URM buildings: a) Two-way spanning, evenly distributed wall arrangement, b) One-way spanning, simple cross-wall structure

(Hendry et al., 1997)

2.5.5.2 Load Transfer

2.5.5.2.1 Wall-to-diaphragm ties/anchors

The lack of adequate detailing to transfer seismic forces from the structure (diaphragms to walls) to the foundation is considered to be the most significant seismic deficiency in URM buildings (FEMA, 2006). Out-of-plane failures or separation of URM walls from their diaphragms often occur from missing or insufficient ties. A rule of thumb for tying diaphragms to walls is that the ties/anchors should be spaced at a distance not exceeding three times the length of the tie or anchor. In addition, unless the ties/anchors are 4 feet on center or less, they are not considered effective in terms of seismic performance (ATC, 2002). In retrofitted buildings, tie failure may occur from bond failure between the masonry units and the grout or pull-through separation of the entire anchor from the wall. Wall-diaphragm failures results from thin walls, poor mortar conditions and the absence of a sufficient shear transfer between the diaphragms and URM walls. In some cases, damage at the corner of URM walls occurs due to the diaphragm finding support by pushing the URM walls in the transverse direction because it cannot transmit the in-plane shear forces to the wall.

2.5.5.3 Component Detailing

2.5.5.3.1 Wall detailing

URM buildings are unreinforced by definition and they do not conform to current seismic design and detailing requirements making them non-ductile, brittle structures. URM walls are weak in out-of-plane bending due to their non-ductile nature and therefore vulnerable when subjected to lateral forces. The vulnerability of a URM wall is a function of the slenderness of the wall. Solid walls can be vulnerable but have the advantage of being less slender. Often, URM buildings with multi-wythe walls containing a cavity (such as two single brick thick walls separated by a gap and connected by metal ties) are slender and behave independently due to missing or badly deteriorated ties or improper bonding along their collar joints (discontinued mortar). The addition of wall-to diaphragm anchors serves to reduce the vertical slenderness of a URM wall as well as make the building work in a homogeneous manner rather than as independent components of the overall structure (EERI, 2006; Ingham & Griffith, 2011a).

2.5.5.3.2 Non-structural elements

Non-structural elements in a URM building pose a major falling hazard to passers-by. These elements often project above and beyond the building roof such as parapets and chimneys. As Figure 2-14 demonstrates for URM parapets of 238 buildings in the 2011 Christchurch earthquake, non-structural elements behave poorly if inadequately anchored/restrained to the overall structure which consequently fail out-of-plane in a brittle, flexural manner when subject to lateral force (FEMA, 2006; Ingham and Griffith, 2011b). In order to diminish the risk of damage or injury these URM elements are properly braced or completely removed from the structure as often they only serve architectural/decorative purposes. Failure of non-structural elements can result from excessive seismic accelerations which cause connection failures between the non-structural elements and the structure; the vulnerability of non-structural elements include cracking, spalling and pounding against adjacent buildings and can lead to localized falling hazards (ATC, 2011; FEMA, 2006).

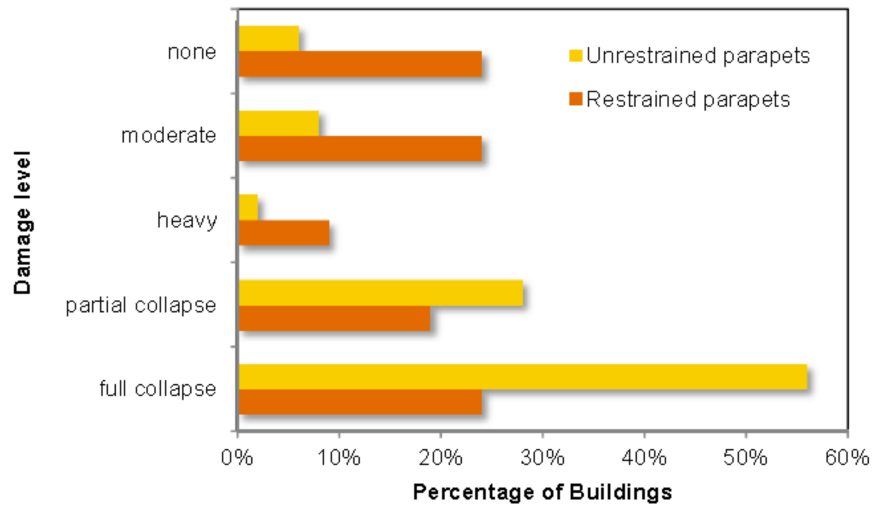


Figure 2-14: Performance of unrestrained and restrained parapets

(Ingham and Griffith, 2011b)

2.5.5.3.3 Quality of construction materials

The quality of construction materials in URM buildings greatly impacts the resiliency of the overall building. An important issue for the in-plane capacity is the relative strength of the masonry units and the mortar quality. In older URM construction, a lime/sand mix was used to bond the units together. Over the years, cement mix was added and the quality of the mortars improved. When the mortars are stronger than the masonry units, the strength of the overall structure may be enhanced. However, brittle cracking through the masonry units may be more likely to occur, resulting in lower deformation capacity (ATC, 1998). In addition, the absence of proper collar joints between masonry units may often lead to an out-of-plane failure or sliding of parts of a URM wall. Generally, poor workmanship in conjunction with poor quality construction materials increases the seismic vulnerability of a URM building.

2.5.5.4 Diaphragms

2.5.5.4.1 Diaphragm type

The diaphragm type of a URM building plays a role in the overall dynamic behavior of the building. In flexible diaphragms, such as timber flooring, higher later deflections can occur in comparison to more rigid diaphragms, such as hollow concrete plank systems (FEMA, 2006). Excessive defections in flexible diaphragms may lead to contribution of partial failure of an out-of-plane wall, while in rigid diaphragms inadequate interconnection or poor detailing may lead to a discontinued load path for transferring lateral forces. However, it has been observed that rigid diaphragms are generally not significantly damaged whereas flexible timber diaphragms may lack strength and stiffness resulting in poor diaphragm action in a URM building (ATC, 1998; ATC 1998).

2.5.5.4.2 Re-entrant corners

Irregular plan configurations of a building, such as L, H or U shaped configurations, where different wings may oscillate out-of-phase may result in major shear stresses in diaphragms. This deficiency is known as a re-entrant corner irregularity (EERI, 2006). The re-entrant corner is a major irregularity problem found in older URM buildings (ATC, 2002). Large demands at the setback of a building tend to pull apart the diaphragms at the corners conditions. The longer and larger the wings are, the higher torsion and stress concentration. The variations in rigidity between the wings cause differential motion between the two parts of the building, resulting in torsional effects and stress concentrations as seen in Figure 2-15.

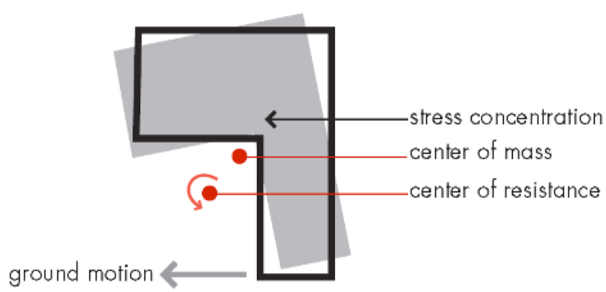


Figure 2-15: Re-entrant corner plan configuration

(EERI, 2006)

2.5.6 Documented URM Performance in Earthquakes

Evidence documented in the performance of structural systems in past earthquakes demonstrates that unreinforced masonry buildings are the most seismically vulnerable construction type. For Example, field observations following the 2011 Christchurch earthquake illustrated in Figure 2-16, identified almost 50% of URM buildings as unsafe to enter (tagged as red) and required further evaluation for safety purposes in comparison to lower values for other building types. Smyrou et al. (2011) documented building damage to structures in the 2010 Darfield and 2011 Christchurch earthquakes of New Zealand. The number of red tagged buildings consisted of: 62% URM, 19% RC, 16% RM, 14% timber, and 7% steel, demonstrating the poor performance of brittle URM buildings and ductile performance of steel buildings. Research on the performance of URM buildings in past earthquakes can provide vital information to properly identify the deficiencies associated with URM buildings. Table 2-4 summarizes observations relating to the seismic performance of URM buildings in previous earthquakes found in New Zealand, United States, Mexico, Australia and Canada.

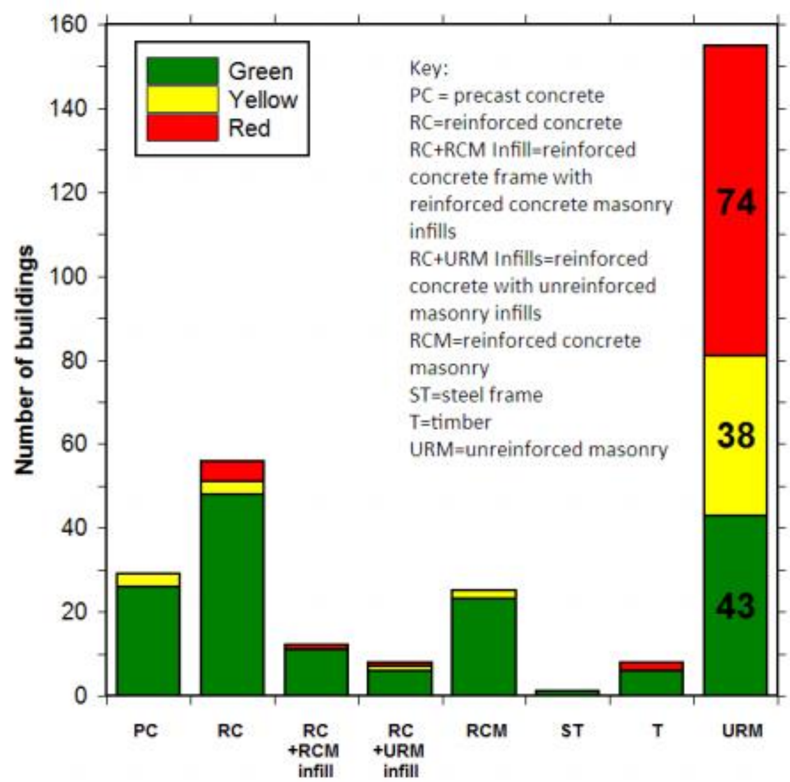


Figure 2-16: Summary of Building Observations in the 2011 Christchurch Earthquake

(Ingham et al., 2011)

Table 2-4: Documented Performance of URM buildings in Previous Earthquakes

Event	Mw	General Comments	Sources
2011 Christchurch (New Zealand)	6.2	• 97% of URM buildings that received no prior strengthening were either seriously damaged or collapsed. • 63% of all URM buildings in the Central Business District (CBD) had received some form of earthquake strengthening • 44% of restrained parapets failed, compared with failure of 84% of unrestrained parapets. • 57% of restrained gable end walls failed compared with 88% of unrestrained gable end walls. • Out-of-plane failure of the veneer with a cavity to the structural wall supporting it was typically attributed to either the deteriorated condition of the metal ties or to pull-out of the ties from the mortar bed joints due to the use of weak lime mortar during construction.	(Ingham and Griffith, 2011a) (Ingham and Griffith, 2011b)
2010 Darfield (New Zealand)	7.1	• City established that they have approximately 7,600 earthquake prone buildings, of which 958 were thought to be constructed of URM. • Complete and partial out-of-plane URM wall failures due to poor (or no) anchorage of wall to diaphragm, some in-plane deformation (cracks passing vertically through lintels), diaphragm deformations, return wall separation, pounding. • Numerous parapet and gable end wall failures observed along both the building frontages and along their side walls. The main failure types observed were: parapet failure, chimney failure, out-of-plane facade wall failure and in-plane damage. • Water ingress had a significant effect on mortar deterioration.	(Dizhur et al., 2010)
2003 Tecoman (Mexico)	7.8	• 17 fatalities and 500 casualties. 15,000 structures damaged (including 3,000 that were destroyed). • Out-of-plane wall failure due to lack of mechanical connection between top of wall and roof or floor diaphragm, combined with inadequate out-of-plane strength. • In-plane shear failure occurring separately and/or in combination with out-of-plane failure. Combined in- and out-of-plane failure often lead to collapse of walls and structures.	(Klingner, 2004)

Event	Mw	General Comments	Sources
2001 Nisqually (United States)	6.8	• URM buildings built before 1950 exhibited poorest behaviour. • Most common damage included shedding of brick from parapets and chimneys. • Other URM buildings exhibited diagonal "stair-step" cracking in walls and piers, damage to walls in the upper stories, vertical cracking in walls, damage to masonry arches, and damage to walls as a result of pounding. • In many cases, fallen brick resulting in damage to objects, such as cars and canopies, outside the building	(ATC, 2009) (Cassidy et al., 2010)
1994 Northridge (United States)	6.7	• 30 fatalities and over 5,000 casualties. Damage estimates ranged from \$15 – 30 billion. • At the time of the earthquake, all URM buildings in the City of Los Angeles had their parapets either removed or laterally braced, and about 80% of URM had been retrofitted. • 2.5% of retrofitted buildings were damaged over 10%, and 0.3% were damaged over 50%. • For non-retrofitted buildings, 10.5% were damaged over 10%, and 7% had damage over 50%.	(Klingner, 2004) (Hess, 2008) (Bruneau, 1995)
1989 Newcastle (Australia)	5.6	• 13 fatalities and over 160 casualties; damaged approximately 50,000 buildings (80% were homes) with URM buildings most widely affected	(Ingham and Griffith, 2011b)
1989 Loma Prieta (United States)	7.1	• 63 fatalities and 3,757 casualties; 12,053 persons displaced; damage and business interruption estimates as high as \$10 billion. • 374 of 2,400 URM in region were vacated due to severe damage (most severely hit building type) URM Damage: • Severe diagonal cracking in columns between URM buildings • Loss of masonry walls improperly tied to the rest of the building • Roof joists or beams slipping off supporting URM walls • Fallen parapets, cornices, exterior cladding/glazing or veneers, and/or decorative elements • Failure of the building as a whole due to insufficient lateral load resistance of URM • Pounding of adjacent buildings	(Bruneau, 1990)
1988 Saguenay (Quebec, Canada)	6.0	• Isolated cases of property damage, mostly non-structural in nature. • Out-of-plane collapse of a gable at the Hippodrome de Quebec.	(Bruneau, 1994b)
1983 Coalinga (United States)	6.2	• 36 of 37 URM buildings damaged – 60% damaged to the extent of having more than half of the walls ruined up to complete collapse	(ATC, 2009)

Event	Mw	General Comments	Sources
1983 Borah Park (United States)	7.3	• Challis, Idaho – two deaths from URM wall failure • Mackay, Idaho – all main street buildings made of unreinforced brick, concrete block or stone were damaged (given size of town, amount of damage constituted a large disaster)	(ATC, 2009)

1933 Long Beach (United States)	6.4	• Following observations of the earthquake, new earthquake regulations in the US building code were implemented (none existed prior)
		• URM construction banned in 1933 following the earthquake
		• 54% of URM displayed significant damage
		  o 20% of cases, damage observed to more than half the wall area, partial collapse, or complete collapse
		  o Many URM buildings were found on school campuses
		• Turning point for URM in the US
			(ATC, 2009)
			(Dizhur et al., 2010)

1931 Napier (New Zealand)	7.8	• Most URM in city's central business district collapsed completely • 260 people killed, URM construction declined following the event • Subsequently, building codes were developed and updated with special attention to performance existing building in earthquakes • Turning point for URM in New Zealand	(Russell et al., 2010) (Dizhur et al., 2010)
1925 Santa Barbara (United States)	6.2	• 40% of URM were severely damaged or collapsed • Most severe damage observed from commercial and residential URM buildings • Motivation for US to adapt seismic design ideas from Japan	(ATC, 2009)
1906 San Francisco (United States)	7.8	• 700-800 deaths • $400 million in direct economic losses • Large stock of URM that were highly vulnerable	(Kircher et al., 2006)

2.6 Summary from Literature Review

The following conclusions can be drawn from the review presented in this chapter:

- Seismic risk is a function of site seismic hazard, building vulnerability and building importance/exposure. These factors are integrated to quantify seismic risk in CanRisk;
- The seismic risk assessment of densely populated areas requires intense collection of building data. The use of GIS can facilitate rapid collection of building data;

- Various seismic risk assessment tools and procedures have been proposed in the literature, including: empirical/statistical models, heuristic models and analytical/mechanistic/theoretical models. The literature review provided a summary of existing methodologies proposed in various countries for the seismic risk assessment of buildings in general as well as tools which are specific to URM;
- The evaluation of the seismic risk is complex and involves uncertainty; a review of fuzzy set theory and fuzzy rule based modelling as method to handle the subjectivity and uncertainty in the evaluation process was presented.
- The literature review provided a detailed state-of-the-art review of the seismic vulnerability of URM buildings. The review included a discussion on URM building characteristics, a review of code development related to URM, a discussion on known seismic deficiencies related to URM and a summary of past performance of URM in documented earthquakes.

The review clearly highlighted the seismic vulnerability of the URM building type. Given the large inventory of URM buildings in Ottawa and other densely populated urban areas in Canada, there is a need to collect building data and develop tools to conduct seismic risk assessments of URM buildings.

Chapter 3
Seismic Risk Assessment of URM/RC Buildings using Hierarchal Fuzzy Rule Base Modeling

3.1 General

This chapter outlines background information that is relevant for the development of seismic risk assessment tool (CanRisk) for buildings. The methodology and implementation of the model is presented. Detailed descriptions of the assessment parameters required for the evaluation are provided. Appendix A provides a user's manual with an example of how to conduct a seismic risk assessment using CanRisk.

3.2 Uncertainty and Complexity

Over the past century, our understanding of natural hazards, such as earthquakes, and the social/economic risks they pose have improved significantly. However, the complexity of seismic events is not always easily understood and almost always contains a form of uncertainty such as the response of our built environment to earthquakes. By means of human reasoning, we are able to predict the outcome using knowledge, intuition and information in order to reduce the complexity and uncertainty involved in any system or situation as illustrated in Figure 3-1. For instance, not until research efforts in the early 20th century, specific information to describe the dynamic behaviour of earthquakes and the significance they have on our built environments was not well understood. This can be described as a case of ignorance, where complexity and uncertainty are abundant. As information becomes readily available, such as collecting data observed from past earthquakes, we are able to reduce the amount the uncertainty and complexity of the problem, such as identifying areas where earthquakes are frequent. As information, rules and algorithms become available to understand and provide essential characteristics of a system, such as location of tectonic plates, performance of different structural systems, soil conditions, mathematical algorithms and human reasoning, does the issue become manageable. Although some deterministic approaches and resources are available to mitigate the effects of earthquakes, such as those in the National Building Code of Canada

(NBCC), the Canadian Standards Association (CSA) design standards, as well as design guidelines outlined in numerous research publications, it is not possible to avoid the vagueness and ambiguity that is inherent in the evaluation of seismic risk. (Ross, 2004; FEMA, 2007)

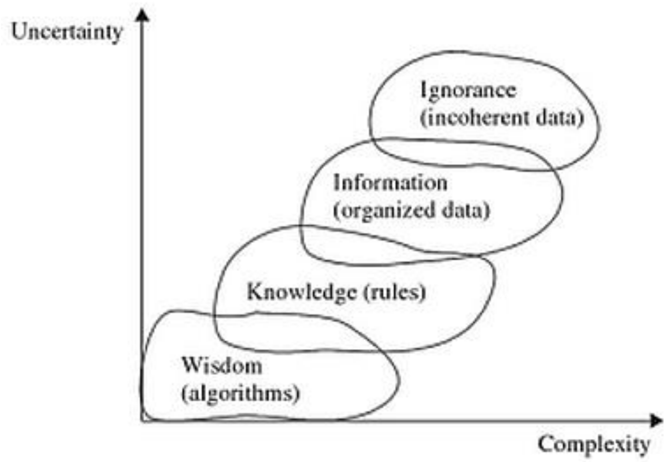


Figure 3-1: Understanding Uncertainty and Complexity

(Ross, 2004)

3.3 Fuzzy Logic Soft Computing Methodology

The use of fuzzy logic for purposes of approximate reasoning has become extensively utilized in engineering applications. The established mathematical procedures embedded in fuzzy logic allows for linguistic, perceptual and qualitative attributes of a system to be translated into numerical reasoning (Tesfamariam, 2008). A fuzzy synthetic evaluation (FSE) is a system involving multiple inputs (e.g. building type, year of construction, site seismic hazard, etc…) aggregated to provide a single outcome/output (e.g. building damageability). The FSE process involves 4 basic steps as indicated below and displayed in Figure 3-2:

Step 1: Identify, collect and quantify input variables essential for the evaluation
Step 2: Establish membership functions for fuzzification of input variables
Step 3: Establish rule-based knowledge and inference implication method
Step 4: De-fuzzification to a single crisp output

As illustrated in the grey area of Figure 3-2, the fuzzy synthetic evaluation contains three main components: membership functions for the fuzzification of values, rule-based inference engine to combine parameters and finally a de-fuzzification method to convert fuzzified values into a single crisp output. The following components are further detailed below with examples relevant to the CanRisk program for further clarification.

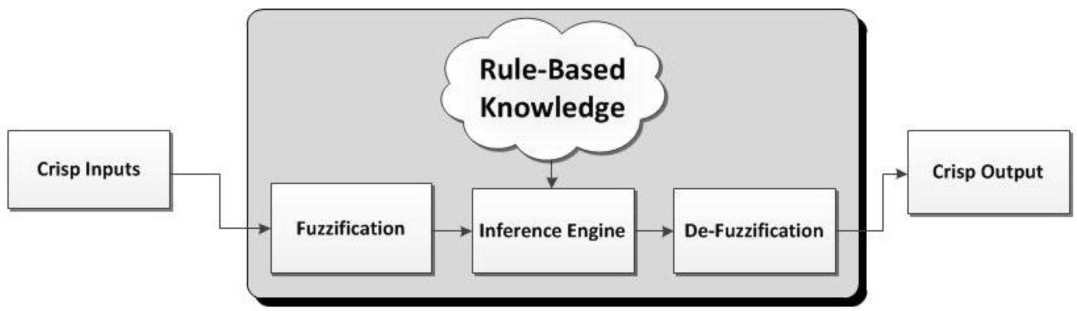


Figure 3-2: General Rule-Based Fuzzy Logic System

3.3.1 Membership functions

In order to build a fuzzy system we must define all the input variables and the adjectives/values that describe them. A set of membership functions are assigned for each input parameter in the fuzzy system. This set of functions defines the parameter's overall influence on the system independently from other parameters. A membership function defines how each input variable is mapped to a membership value (or a degree of membership) (MathWorks, 1999). For instance, the purpose of assigning membership functions is to fuzzify an input variable (e.g. spectral acceleration) and assigned partial membership values in a set (e.g. low, moderate, high hazard degrees of membership). An unlimited amount of variations in forming membership functions exist that are used to describe a parameter such as cubic or bell-shaped expressions, but the most common membership functions are triangular and trapezoidal membership functions (Castro, 1995). The triangular fuzzy numbers (TFN) or trapezoidal fuzzy numbers (TPFN) define the coordinates and interval of each membership function in a set. Subsequently, Figure 3-3 represents the fuzzy set of membership functions established for determining the degrees of membership value for a level of vertical irregularity (VI). The example illustrated in Figure 3-3, contains 5 triangular membership functions to represent the vertical irregularity parameter. They are expressed as {*Very low; Low; Medium; High; Very high*} with respective triangular fuzzy numbers of [TFN(0,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1.0,1.0)] which define the intervals for each function. For example, if an engineer assesses a building to have a *very high* vertical irregularity resulting from a soft story on the first floor. The fuzzy system converts the linguistic description into a transformation value from scale of 0 to 1. The *very high* vertical irregularity transforms into 0.9 on the x-axis of Figure 3-3. The intersecting membership functions deliver the resultant degrees of membership of the assessment. Therefore a *very high* vertical irregularity's fuzzy values are ($\mu_{VL}^{VI} = 0.00$, $\mu_{L}^{VI} = 0.00$, $\mu_{M}^{VI} = 0.00$, $\mu_{H}^{VI} = 0.40$, $\mu_{VH}^{VI} = 0.60$). In other words, a *very high* vertical irregularity has a 40% degree membership in high and a 60% degree membership in very high as defined by the membership functions of the parameter.

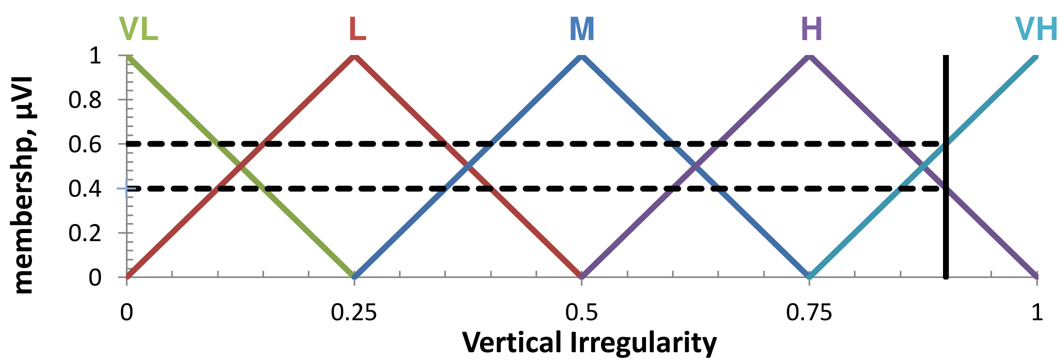


Figure 3-3: Vertical Irregularity Fuzzy Set of Membership Functions

The aforementioned membership functions can be formed by using straightforward techniques to assign membership values to form the functions such as using intuition, inference, rank ordering, inductive reasoning and algorithmic or logical operations (Ross, 2004).

3.3.2 Rule-Based Knowledge / Inference Engine

Once the membership functions have been formed and the crisp inputs have been fuzzified to obtain degrees of membership for each independent parameter as seen in section 3.3.1, the combination and relationship of parameters and their respective degrees of membership values are evaluated through fuzzy rule-based (FRB) expressions (e.g. if building vulnerability is low and seismicity is low, building damageability is low). These lists of if-then statements refer to the fuzzy rule-based knowledge. The rule-based knowledge introduces non-linearity in the overall system where the rules are evaluated in parallel meaning the sequence of rules is insignificant (MathWorks, 1999).

The type of inference engine applied in a fuzzy system expresses how two inputs (also called the antecedents) infers or derives the output or conclusion (the consequent) of the system based on the expression on rules. The most common inference engine for fuzzy systems is the Mamdani implication method (Castro, 1995; Ross, 2004). Accordingly, Mamdani inference methodology is applied to the rule-based expressions of the CanRisk program. For example, if we have a system of two independent inputs defined as X_1 and X_2 and a single output denoted as Y for a collection of N number of linguistic IF-THEN expressions, the Mamdani inference engine is represented as Equation 3-1 (Tesfamariam, 2008):

$$\mathbf{R_i : IF}\ X_1 \text{ is } A_{i1}\ \mathbf{AND}\ X_2 \text{ is } A_{i2}\ \mathbf{THEN}\ Y \text{ IS } B \text{ for } i=1,2,\ldots K \qquad [3\text{-}1]$$

Where:

- R_i represents the i^{th} rule
- X_1 and X_2 are the input variables (antecedents)
- K is the total number of rules
- A_{i1} and A_{i2} are the input membership fuzzy sets
- Y is the output variable (consequent)
- B is the output membership fuzzy set

Lastly, the combinations of rules require an aggregation operator in order to provide a single fuzzy output value. In the Mamdani system type, the most common aggregation operators are minimum and maximum operators where two sets of fuzzy values (X_1 and X_2) result into a single fuzzy value (Y) (Tamás and Kóczy, 2007). The following example illustrates the use of the Mamdani inference engine to a set of rule-based expressions.

Example: From an evaluation of a building, the presence of vertical irregularity (VI) and plan irregularity (PI) are identified as *very high* and *low*, respectively. The aggregation of the two inputs results in the increase in seismic demand (ID). Given the fuzzified values of VI and PI as ($\mu_{VL}^{VI} = 0.00$, $\mu_{L}^{VI} = 0.00$, $\mu_{M}^{VI} = 0.00$, $\mu_{H}^{VI} = 0.40$, $\mu_{VH}^{VI} = 0.60$) and($\mu_{VL}^{PI} = 0.00$, $\mu_{L}^{PI} = 0.80$, $\mu_{M}^{PI} = 0.20$, $\mu_{H}^{PI} = 0.00$, $\mu_{VH}^{PI} = 0.00$) respectively. Table 3-1 defines the FRB established in order to determine the degrees of membership for the ID of a building. The resultant crisp value for the ID can be determined by evaluating Equation 3-1. Finally, the minimum operator is applied for each rule in order to determine the degree of membership for the subsequent rule.

Table 3-1: Example Rule-based Evaluation for the Increase in Demand of a Building

$R_{1:}$	**IF**	VI	$\mu_{VL}^{VI} = 0$	**AND**	PI	$\mu_{VL}^{PI} = 0$	**THEN**	ID	$\mu_{VL}^{ID} = \min(0,0)=0$
$R_{2:}$	**IF**	VI	$\mu_{VL}^{VI} = 0$	**AND**	PI	$\mu_{L}^{PI} = 0.8$	**THEN**	ID	$\mu_{VL}^{ID} = \min(0,0.80) = 0$
$R_{3:}$	**IF**	VI	$\mu_{VL}^{VI} = 0$	**AND**	PI	$\mu_{M}^{PI} = 0.2$	**THEN**	ID	$\mu_{L}^{ID} = \min(0,0.2) = 0$
$R_{4:}$	**IF**	VI	$\mu_{VL}^{VI} = 0$	**AND**	PI	$\mu_{H}^{PI} = 0$	**THEN**	ID	$\mu_{L}^{ID} = \min(0,0) = 0$
$R_{5:}$	**IF**	VI	$\mu_{VL}^{VI} = 0$	**AND**	PI	$\mu_{VH}^{PI} = 0$	**THEN**	ID	$\mu_{M}^{ID} = \min(0,0) = 0$
$R_{6:}$	**IF**	VI	$\mu_{L}^{VI} = 0$	**AND**	PI	$\mu_{VL}^{PI} = 0$	**THEN**	ID	$\mu_{VL}^{ID} = \min(0,0) = 0$
$R_{7:}$	**IF**	VI	$\mu_{L}^{VI} = 0$	**AND**	PI	$\mu_{L}^{PI} = 0.8$	**THEN**	ID	$\mu_{L}^{ID} = \min(0,0.8) = 0$
$R_{8:}$	**IF**	VI	$\mu_{L}^{VI} = 0$	**AND**	PI	$\mu_{M}^{PI} = 0.2$	**THEN**	ID	$\mu_{L}^{ID} = \min(0,0.2) = 0$
$R_{9:}$	**IF**	VI	$\mu_{L}^{VI} = 0$	**AND**	PI	$\mu_{H}^{PI} = 0$	**THEN**	ID	$\mu_{M}^{ID} = \min(0,0) = 0$
$R_{10:}$	**IF**	VI	$\mu_{L}^{VI} = 0$	**AND**	PI	$\mu_{VH}^{PI} = 0$	**THEN**	ID	$\mu_{M}^{ID} = min(0,0) = 0$
$R_{11:}$	**IF**	VI	$\mu_{M}^{VI} = 0$	**AND**	PI	$\mu_{VL}^{PI} = 0$	**THEN**	ID	$\mu_{L}^{ID} = \min(0,0) = 0$
$R_{12:}$	**IF**	VI	$\mu_{M}^{VI} = 0$	**AND**	PI	$\mu_{L}^{PI} = 0.8$	**THEN**	ID	$\mu_{L}^{ID} = \min(0,0.8) = 0$
$R_{13:}$	**IF**	VI	$\mu_{M}^{VI} = 0$	**AND**	PI	$\mu_{M}^{PI} = 0.2$	**THEN**	ID	$\mu_{M}^{ID} = \min(0,0.2) = 0$
$R_{14:}$	**IF**	VI	$\mu_{M}^{VI} = 0$	**AND**	PI	$\mu_{H}^{PI} = 0$	**THEN**	ID	$\mu_{M}^{ID} = \min(0,0) = 0$
$R_{15:}$	**IF**	VI	$\mu_{M}^{VI} = 0$	**AND**	PI	$\mu_{VH}^{PI} = 0$	**THEN**	ID	$\mu_{H}^{ID} = \min(0,0) = 0$
$R_{16:}$	**IF**	VI	$\mu_{H}^{VI} = 0.4$	**AND**	PI	$\mu_{VL}^{PI} = 0$	**THEN**	ID	$\mu_{L}^{ID} = \min(0.40,0) = 0$
$R_{17:}$	**IF**	VI	$\mu_{H}^{VI} = 0.4$	**AND**	PI	$\mu_{L}^{PI} = 0.8$	**THEN**	ID	$\mu_{M}^{ID} = \min(0.4,0.8) = 0.4$
$R_{18:}$	**IF**	VI	$\mu_{H}^{VI} = 0.4$	**AND**	PI	$\mu_{M}^{PI} = 0.2$	**THEN**	ID	$\mu_{H}^{ID} = \min(0.4,0.2) = 0.2$
$R_{19:}$	**IF**	VI	$\mu_{H}^{VI} = 0.4$	**AND**	PI	$\mu_{H}^{PI} = 0$	**THEN**	ID	$\mu_{H}^{ID} = \min(0.4,0) = 0$
$R_{20:}$	**IF**	VI	$\mu_{H}^{VI} = 0.4$	**AND**	PI	$\mu_{VH}^{PI} = 0$	**THEN**	ID	$\mu_{VH}^{ID} = \min(0.4,0) = 0$
$R_{21:}$	**IF**	VI	$\mu_{VH}^{VI} = 0.6$	**AND**	PI	$\mu_{VL}^{PI} = 0$	**THEN**	ID	$\mu_{M}^{ID} = \min(0.6,0) = 0$
$R_{22:}$	**IF**	VI	$\mu_{VH}^{VI} = 0.6$	**AND**	PI	$\mu_{L}^{PI} = 0.8$	**THEN**	ID	$\mu_{M}^{ID} = \min(0.6,0.8) = 0.6$
$R_{23:}$	**IF**	VI	$\mu_{VH}^{VI} = 0.6$	**AND**	PI	$\mu_{M}^{PI} = 0.2$	**THEN**	ID	$\mu_{H}^{ID} = \min(0.6,0.2) = 02$
$R_{24:}$	**IF**	VI	$\mu_{VH}^{VI} = 0.6$	**AND**	PI	$\mu_{H}^{PI} = 0$	**THEN**	ID	$\mu_{VH}^{ID} = \min(0.6,0) = 0$
$R_{25:}$	**IF**	VI	$\mu_{VH}^{VI} = 0.6$	**AND**	PI	$\mu_{VH}^{PI} = 0$	**THEN**	ID	$\mu_{VH}^{ID} = \min(0.6,0) = 0$

Finally, applying the maximum operator and aggregating the above rules we obtain the membership values for the increase in demand as seen in Table 3-2:

Table 3-2: Aggregation Process of Rules for the Increase in Demand of a Building

μ_{VL}^{ID} = max(0,0,0)=0
μ_{L}^{ID} = max(0,0,0,0,0,0,0)=0
μ_{M}^{ID} = max(0,0,0,0,0,0.4,0,0.6)=0.6
μ_{H}^{ID} = max(0,0,0.2,0.2)=0.2
μ_{VH}^{ID} = max(0,0,0)=0

Therefore a *very high* vertical irregularity and *low* plan irregularity of a building results in an increase in demand of the system with 60% membership in medium and 20% membership in high for the ID resulting from the evaluated FRB expressions in Table 3-1.

3.3.3 De-fuzzification

The final step in the fuzzy evaluation after aggregating all the membership values of an output is the de-fuzzification process. De-fuzzification involves evaluating the fuzzy values that result from the membership functions and rule-base in order to generate a single, crisp output. The center of area is the most common de-fuzzification method (Castro, 1995) and is expressed in Equation [3-2.

Area of Centroid:

$$Centroid = \frac{\sum C_{i_x} A_i}{\sum A_i} \qquad [3\text{-}2]$$

Where:

- C_{i_x} represents the centroid of a shape along the x-axis
- A_i is the area of the shape

Continuing the example conducted using the Mamdani type inference engine, the resultant fuzzy values for the increase in demand (ID) are ($\mu_{VL}^{ID} = 0.00$, $\mu_{L}^{ID} = 0.00$, $\mu_{M}^{ID} = 0.60$, $\mu_{H}^{ID} = 0.20$, $\mu_{VH}^{ID} = 0.00$) as depicted in Table 3-2. Thus, the resultant shape of the fuzzified values for the ID is outlined in Figure 3-4.

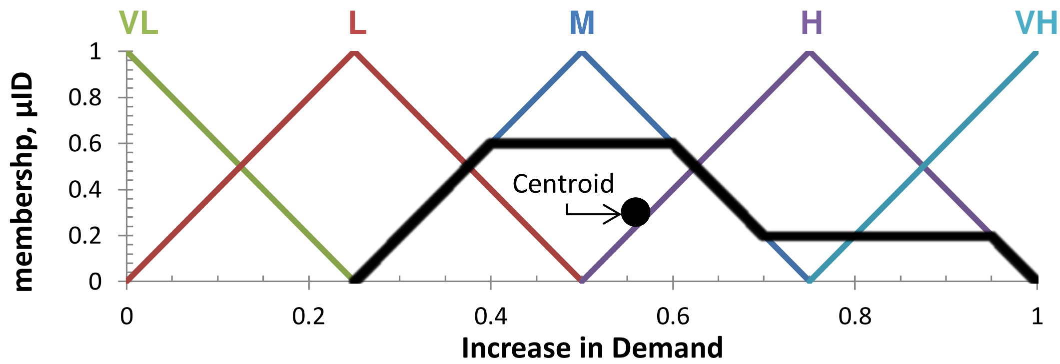


Figure 3-4: Center of Area Calculation for Increase in Demand from resulting VI and PI Input variables.

Finally, applying the center of area equation, Equation [3-2, we obtain an increase in demand of:

$$Centroid = \frac{\sum C_{i_x} A_i}{\sum A_i} = 0.56$$

Therefore, the increase in demand of the evaluation has a value of 0.56 as represented in Figure 3-4, where the value of the ID will be continued in the hierarchical evaluation process.

A 3D surface can be generated for the above fuzzy mathematical procedure in order to visualize the variation of an output value from two input values. Figure 3-5 illustrates the 3D surface for the evaluation of the increase in demand (ID) of a structure.

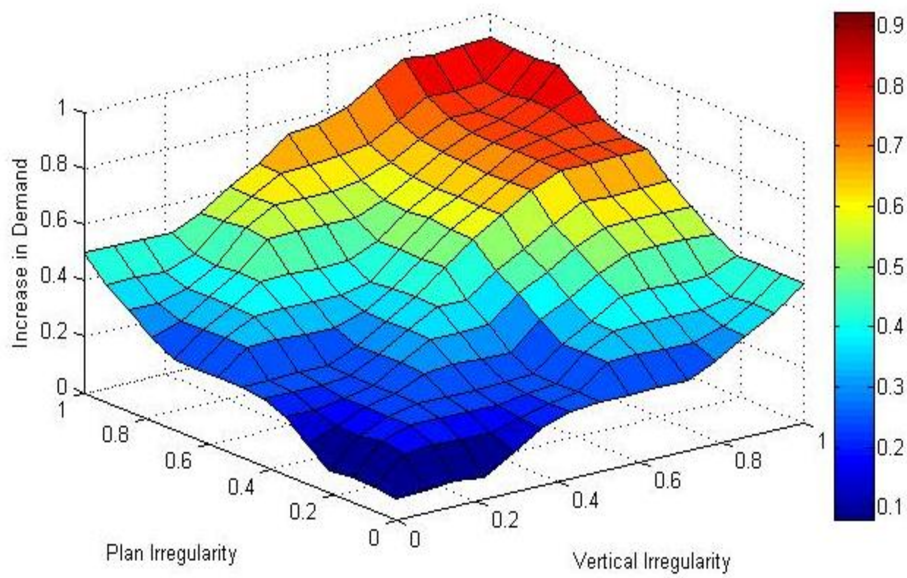


Figure 3-5: 3D Surface for the Increase in Demand Evaluation of a Building

3.4 Hierarchical structure for seismic risk analysis of buildings

As mentioned previously, fuzzy logic deals with mathematical modeling of vague concepts such as evaluating seismic risk. The complex problem of risk-based inspection is handled through a simple manageable hierarchical structure as illustrated in Figure 3-6 and Figure 3-7. Each pair of parameters are evaluated using the fuzzy synthetic evaluation (FSE) outlined in Section 3.3 where membership functions are established for each parameter and a rule base is created for the combination of parameters. The hierarchy follows a logical order where the casual relationship for each supporting argument is further subdivided into specific contributors (Tesfamariam, 2008). The hierarchy incorporates three main modules in order to quantify seismic risk: site seismic hazard, building vulnerability (structural and non-structural) and building importance. The building vulnerability contains two new different module types in comparison to the original CanRisk hierarchy for concrete structures as illustrated in Figure 2-1 . The first for the hierarchical structural seismic risk analysis as seen in Figure 3-6 and the second hierarchy representing the non-structural seismic risk analysis as shown in Figure 3-7.

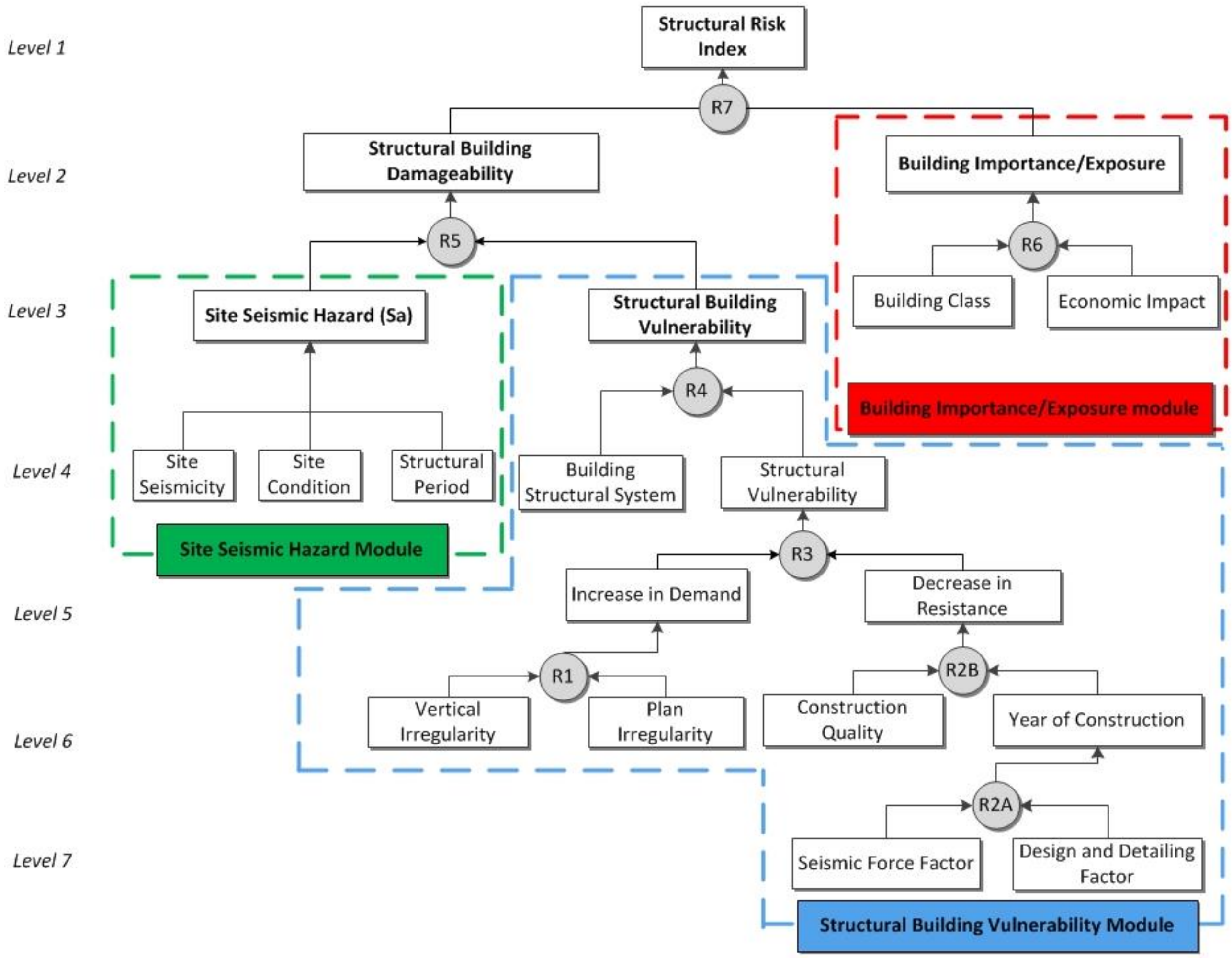


Figure 3-6: Hierarchical Structural Seismic Risk Analysis of URM/RC Buildings

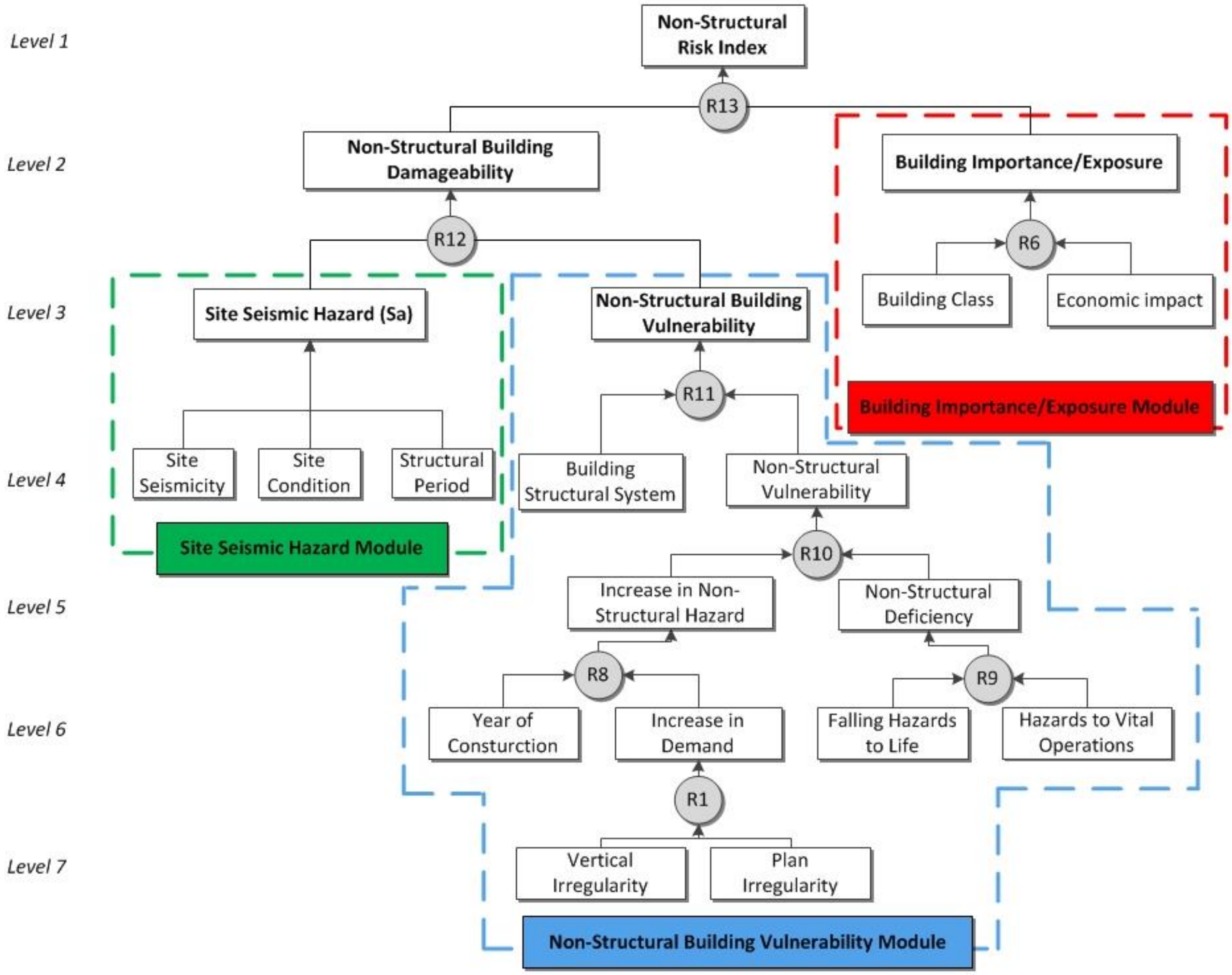


Figure 3-7: Hierarchical Non-structural Seismic Risk Analysis of URM/RC Buildings

The site seismic hazard module integrates structural period, soil type and site seismicity to form the Uniform Hazard Spectra outlined in the 2005 NBCC, which reflects 4^{th} generation seismic hazard values in Canada as also adopted by the 2010 NBCC, and evaluates the spectral acceleration corresponding to structure's period as indicator of site seismic hazard. The building vulnerability captures the inherent system deficiencies arising from structural and non-structural factors as well as the structural system considered in the evaluation. Lastly, the building importance/exposure module assesses the building area, building use, occupancy type and economic impact to quantify expected human loss, emergency response capacity and/or economic loss (Tesfamariam, 2008). The three main modules are further described below.

3.5 Site Seismic Hazard Module (I^{SSH})

Site seismic hazard pertains to the ground motions generated due to earthquakes and the affects they pose on existing built infrastructure. One way of establishing site seismic hazard is to adopt design spectra specified in buildings codes established based on seismic hazard values. In Canada, the design level of earthquake is expressed in terms of the Uniform Hazard Spectra (UHS) defined in the 2010 edition of the NBCC with 2% probability of exceedance in 50 years. The UHS is determined directly as a function of geographical location and site condition and is defined by intervals of structural periods calculated at the same probability of exceedance (Heidebrecht, 2003). In order to formulate the UHS of a specific building, three main parameters are considered, consisting of site seismicity, site condition and the fundamental period of vibration of a structure. The overall fuzzification process of a building's spectral acceleration is briefly summarized in Figure 3-8.

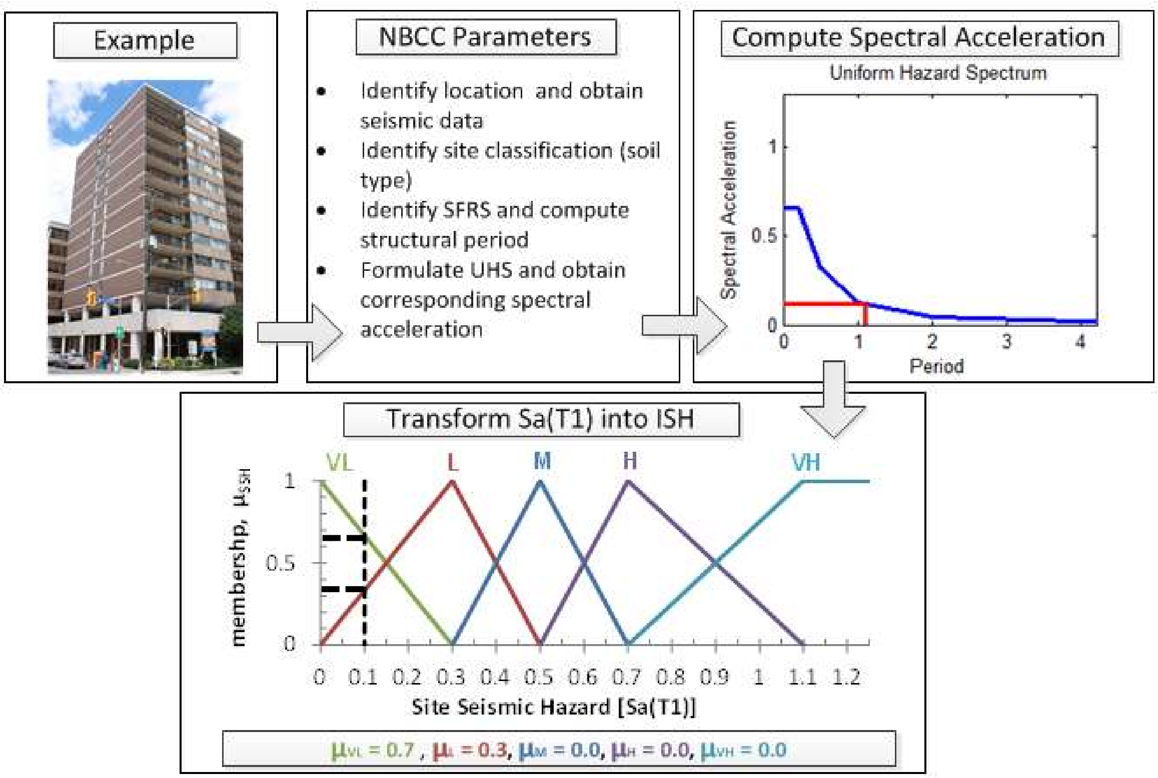


Figure 3-8: Fuzzification Process of Site Seismic Hazard using Spectral Acceleration

3.5.1 Site Seismicity

Site seismicity reflects the level of seismic hazard in a given geographic location. The NBCC-2010 provides seismic data based on location for 0.2, 0.5, 1.0 and 2.0 second periods. Linear interpolation is used for in-between values and the shape of the Uniform Hazard Spectra (UHS) is constructed for 5%-damped spectral accelerations, with a probability of exceedance of 2% in 50 years (Atkinson, 2004). Table 3-3 lists the spectral acceleration values for Edmonton, Alberta; Ottawa, Ontario; and Victoria, British Columbia, which are considered to be regions of low, moderate and high seismicity, respectively.

Table 3-3: NBCC-2010 Seismic Data for Edmonton, Ottawa and Victoria

Location	Seismic Data				
	Sa(0.2)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**PGA**
Edmonton, Alberta	0.095	0.057	0.026	0.008	0.036
Ottawa, Ontario	0.64	0.31	0.14	0.046	0.32
Victoria, British Columbia	1.2	0.82	0.38	0.18	0.61

3.5.2 Site Condition

The site soil conditions through which seismic waves travel affect seismic hazard. The NBCC-2010 (NRCC, 2010) provides a list of site classifications for different soil profiles as presented in Table 3-4. Local ground conditions for a building are a key indicator to ground shaking intensities due to amplification/deamplification of seismic waves during a seismic event. These code provisions result from lessons learned from previous earthquakes, such as the 1989 Loma Prieta earthquake which demonstrated the amplification of ground motions on soft soil sites (Heidebrecht, 2003), as well as soil tests. They are based on shear wave velocities of soils. In general, hard rock and rock ground conditions (soil profiles A and B) display a lower amplitude in ground motion intensity in comparison to stiff or soft soil ground conditions (soil profiles D and E), where the amplification of seismic waves increases the likelihood of damage due to an increase in ground shaking (Motazedian et. al., 2011).

Table 3-4: NBCC-2010 Site Classification

Site Class	Profile Name	Shear Wave Velocity (V_s)
A	Hard rock	$V_s > 1500$
B	Rock	$750 < V_s \leq 1500$
C	Very dense soil and soft rock	$360 < V_s < 760$
D	Stiff Soil	$180 < V_s < 360$
E	Soft Soil	$V_s < 180$

3.5.3 Fundamental Period of Vibration

The final step to determine site seismic hazard is to evaluate the building's fundamental period of vibration (T_1). The period of a building is the inverse of the natural frequency of vibration of a building when set into motion. It is an important design parameter used in the computation of base shear for a given building. Buildings with shorter periods will attract higher seismic forces as seen on the UHS. The NBCC provides empirical formulas to provide approximate structural periods based on the seismic force resisting system (SFRS) of a building as listed in Table 3-5. However, the NBCC allows other established methods of mechanics to determine structural periods but places an upper limit to avoid overestimating the period (Saatcioglu and Humar, 2003). The equations in Table 3-5 are based on the regression analysis of data recorded during previous seismic events. Once the structural period is determined, the spectral acceleration is established and used as the transformation value to provide a rational indicator of site seismic hazard.

Table 3-5: Empirical Formulas for Building Period defined in NBCC-2010 (NRCC, 2010)

Seismic Force Resisting System (SFRS)	Fundamental Lateral Period (T_1)
Steel Moment Frames	$0.085(h_n)^{3/4}$
Concrete Moment Frames	$0.075(h_n)^{3/4}$
Other Moment Frames	0.1N
Braced Frames	$0.025h_n$
Shear Wall and other structures	$0.05(h_n)^{3/4}$
T_1 (seconds), h_n (meters), N (Number of stories)	

3.6 Building Vulnerability Module (I^{BV})

The building vulnerability module (seen as level 3 in Figure 3-6 and Figure 3-7) captures the inherent structural and non-structural deficiencies of a particular building type. The module is divided into two main vulnerability categories, building structural vulnerability (BSV) and non-structural vulnerability (NSBV) seen as level 4 Figure 3-6 and Figure 3-7, respectively. The structural vulnerability considers the performance modifiers relevant to the structural elements and configuration of the building while the non-structural vulnerability evaluates all the other components that are included in a building but not integrated as part of the structural system. Aggregation of the parameters in the hierarchical structure to the building vulnerability module, highlighted blue in Figure 3-6 and Figure 3-7, are used to compute the structural building vulnerability index ($\mathrm{I}_{\mathrm{S}}^{\mathrm{BV}}$) and non-structural building vulnerability index (I_{NS}^{BV}). The structural building vulnerability and non-structural building vulnerability assessment parameters are further described below.

3.6.1 Structural Vulnerability (SV)

The structural vulnerability of a building indicates characteristic deficiencies in design and construction. Observations from past performance of buildings during earthquakes demonstrate that the structural vulnerability is a key parameter affecting building's seismic vulnerability. For instance, the damage to reinforced concrete and masonry structures in the 1997 Turkey Earthquake, where many buildings collapsed and others experienced light to moderate damage, resulted from structural vulnerabilities attributed to non-ductile buildings (Saatcioglu et al., 2001). The damage was attributed to structural deficiencies such as irregular building layouts and insufficient seismic design and detailing. Structural vulnerability of buildings can be grouped in two main categories: i) factors contributing to increased seismic demands (increase in demand) and ii) factors contributing to reduced ductility and energy absorption capacity (decrease in resistance) (Saatcioglu et al., 2001; Tesfamariam, 2008).

3.6.1.1 Increase in Demand (ID)

The increase in demand (ID) of a building results from structural irregularities that tend to attract more seismic forces and deformations increasing seismic demands on the structure. It isn't until the 2005 edition of the NBCC did the code categorize eight types of structural irregularities resulting in structural damage caused by past earthquakes that engineers and designers are encouraged to avoid in design (NRC, 2005). This edition of the NBCC provides rational treatment to irregularity incorporated into the design of a building and includes limitation to the use of the static analysis procedure and compels a designer to use a dynamic analysis as the preferred method of evaluation for irregular structures. In addition, restrictions of the irregularities allowable are outlined in comparison to the extent of seismic hazard, building importance and seismic design forces (Mitchel et al., 2005; Heidebrecht, 2003; NRCC, 2005). An increase in demand on a structure results from two main different types of irregularity categorized as i) vertical irregularity and ii) plan irregularity (Tesfamariam, 2008; EERI, 2006).

3.6.1.1.1 Vertical Irregularity (VI)

Vertical irregularities (VI) consist of abrupt changes in stiffness (soft story) and strength (weak story) along the building height (Tesfamariam, 2008). For example, a first storey retail space of a building causes a soft-storey vertical irregularity resulting in higher drifts of a building (Saatcioglu et al, 2001). Furthermore, a vertical irregularity can arise from irregular vertical geometric configurations such as setbacks, discontinuity in the lateral force resisting elements (e.g. a shear wall) and irregular mass distributions between storey levels. The aforementioned vertical irregularities are highlighted in the 2010 edition of the NBCC (NRCC, 2010). The severity of a vertical irregularity can best be judged by a structural engineer. The influence of an irregularity varies with the degree of severity, which depends on the extent of the irregularity in the structure (EERI, 2006). In a walk down survey, the level of vertical irregularity can only be determined linguistically, to be transformed and used for fuzzification purposes as seen in Table 3-6. The severity of an irregularity is left to the judgement of the engineer.

Table 3-6: Linguistic Input Parameters and Transformation Values for VI

Linguistic Input	Transformation Value
"Very Low"	0.1
"Low"	0.3
"Moderate"	0.5
"High"	0.7
"Very High"	0.9

3.6.1.1.2 Plan Irregularity (PI)

A plan irregularity (PI) arises from asymmetric plan configurations, such as torsional effects generated by large eccentricities between the center of mass and center of resistance of a structure. Areas of high stress concentrations in plan, such as re-entrant corners, form another example of a plan irregularity (Tesfamariam, 2008; ERRI, 2006). As mentioned previously, the level of irregularity is left to the judgement of the engineer. Therefore, in a walk down survey, the assessment of plan irregularity is determined in a similar manner as the vertical irregularity. Linguistic terms assigned to the levels of plan irregularities are presented in Table 3-7.

Table 3-7: Linguistic Input Parameters and Transformation Values for PI

Linguistic Input	Transformation Value
"Very Low"	0.1
"Low"	0.3
"Moderate"	0.5
"High"	0.7
"Very High"	0.9

3.6.1.1.3 Decrease in Resistance (DR)

The decrease in resistance results from any reduction in capacity of a building due to poor design and construction practices. This may be related to year of construction and the building code in effect, or the level of code enforcement (Saatcioglu et al, 2001; Mitchell et al., 2005). For instance, lack of sufficient ties in concrete columns result in shear failure, and ultimately the failure of the columns. Lack of confinement reinforcement and detailing cause compression concrete to crush in a brittle manner (Saatcioglu et al, 2001). Similarly, for URM buildings, inadequate ties between load-bearing walls and diaphragms may lead to an out-of-plane failure of the wall. Deteriorated URM materials may also affect the lateral force capacity of the building. The decrease in resistance can be established by two performance modifiers: i) the construction quality and ii) the year of construction. The uses of the two performance modifiers in CanRisk are subsequently explained.

3.6.1.1.4 Construction Quality (CQ)

Construction quality (CQ) and the use of quality materials in design play important roles on seismic response of a building. It has been noted that good construction practices and high quality materials in older, more vulnerable buildings can improve seismic performance of buildings (Bruneau and Lamontange 1994; Coburn and Spence 2002). Poor quality concrete or masonry materials, construction errors, improper construction practices, and lack of anchorage can lead to serious consequences as seen in previous eastern Canadian earthquakes (Paultre et al 1993; Bruneau and Lamontagne 1994). The construction quality can be qualitatively evaluated as defined in Table 3-8 (Tesfamariam, 2008).

Table 3-8: Linguistic Input Parameters and Transformation Values for CQ

Linguistic Input	Transformation Value
“Good”	0.1
“Average”	0.5
“Poor”	0.9

3.6.1.1.5 Year of Construction (YOC)

The year of construction (YOC) is an important factor affecting seismic performance of a building. When considering the historical development of NBCC and CSA standards in Canada, the year of construction can provide a significant insight on design loads and the level of detailing relevant to the era during which the building was constructed. Therefore, it is important to understand the evolution of seismic design criteria over the last century in order to determine the seismic vulnerability of buildings.

Seismic Force Factor (SFF)

Increase in scientific knowledge results in improved provisions of building codes (Heidebrecht, 2003). Because building codes provide seismic design force levels, the comparison of equivalent static seismic forces proposed in different editions of NBCC reflects the progression of our knowledge on seismic design over the years. This information is important from the point of view of assessing seismic vulnerabilities of buildings designed in different years, assuming those designed based on the current practice are the least vulnerable. Equation [3-3 can be used to compare the current levels of seismic design forces, which are based on the Canadian seismic hazard values proposed in the 2005 NBCC, with those used in previous years. The NBCC design force levels proposed throughout the years of code development are illustrated in Figure 3-9. Detailed information on the evolution of seismic base shear equations can be found in Mitchell et al., (2010). While the ratio of seismic design forces in different years reflects the vulnerability of buildings designed in different years, especially when seismic forces govern design, this may not be the case for buildings in regions of moderate to low seismicity. In regions of moderate seismicity, the significance of seismic forces on design may be less significant and any change in seismic design force levels over the years may not play as important roles as those in regions of high seismicity. Therefore, the ratio of seismic design forces is then multiplied by the current design spectral acceleration to reflect the significance of earthquake forces on building designs in a given region based on seismicity. The resulting product is used to reflect building vulnerability associated with design force levels. This product is referred to as "Seismic Force Factor."

Seismic Force Factor (SFF):

$$SFF = \frac{V_{2005}}{V_{Previous}} * S_a^c(T_1) = \frac{S_a(T_1)M_V W}{V_{Previous}} * S_a^c(T_1) \qquad [3\text{-}3]$$

Where:

- SFF is the Seismic Force Factor
- V_{2005} is the base shear equation according to NBCC-2005
- $V_{Previous}$ is the base shear equation from Figure 3-9 for the YOC selected
- $S_a^c(T_1)$ is the building's spectral acceleration expressed as a percentage of g at reference soil class C according to NBCC-2005.
- M_V is the higher mode effect factor according to NBCC-2005
- W is the weight of the building

In addition to introducing the effect of seismicity of the region, a set of cut-off values are also assigned to the Seismic Force Factor to control its sensitivity to the base shear ratio. The cut-off values, shown in Table 3-9, are introduced using the seismicity ranges used in Table 4.1.8.9 of the 2010 NBCC based on spectral accelerations at 0.2 sec.

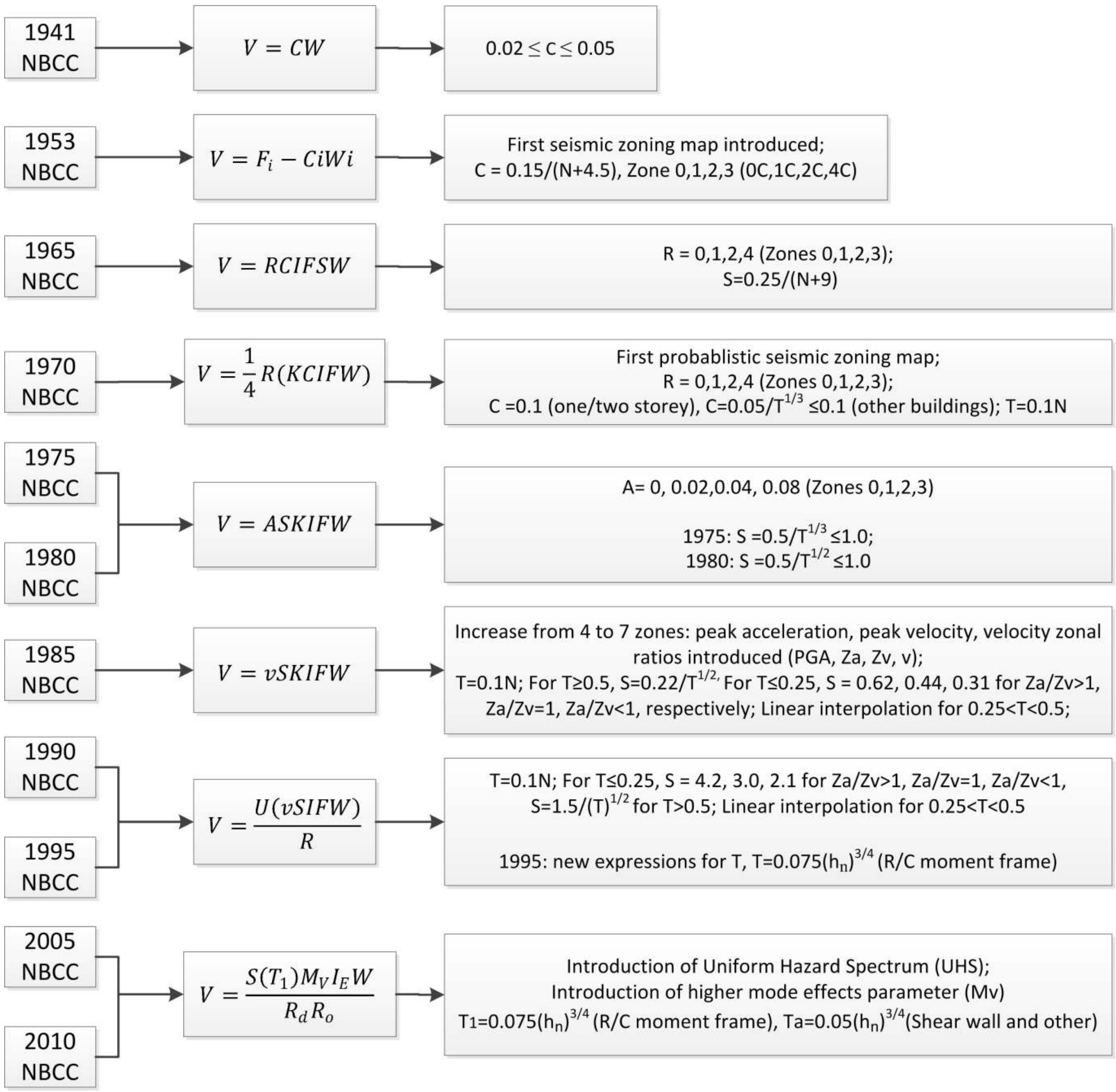


Figure 3-9: Historical Development of Base Shear Equation of the NBCC

Table 3-9: Cut-off values for the ratio of NBCC-2005 base shear to previous year base shear equations

	Value of Sa(0.2)			
Range	< 0.2	≥ 0.2 but <0.35	≥0.35 but ≤0.75	>0.75
Cut-off Value $(\frac{V_{2005}}{V_{Previous}})$	1	2	3	4

Higher mode effects on design spectra were accounted for in 1995 NBBC by raising the spectra artificially in the high period range. The 2005 provisions recognized this effect explicitly by introducing factor M_v. This is accounted for in computing the base shear ratio given in Equation 3-3. Furthermore, the spectral values $S_a(T_1)$ and $S_a^c(T_1)$ used in the same equation include the 2/3 cut-off values assigned to buildings in the short period range when the building has some minimum ductility, which is defined in the 2005 and 2010 NBCC as buildings in categories that correspond to designs with $R_d \geq 1.5$. This implies that the spectral values used in Equation 3-3 has a maximum limit of 2/3 $S_a(0.2)$ for all reinforced concrete buildings. The same cut-off value does not apply to unreinforced masonry buildings, as per 2005 NBCC

Design and Detailing Factor (DDF)

The performance of well detailed and designed buildings in recent earthquakes indicate structural systems exhibiting ductile behaviour outperform poor structural systems that fail in a brittle manner (Saatcioglu et al., 2001). Therefore, earthquake-resistant buildings are designed for strength and ductility. Seismic forces can be reduced when a structure exhibits ductile behaviour or enters into the inelastic range of deformations. The amount of reduction in elastic forces and the level of detailing required to attain the corresponding level of ductility are intimately related. The former is specified in NBCC and the latter is specified in relevant CSA standards. The progression of design and detailing requirements outlined in CSA standards for different materials, such as reinforced concrete, and the introduction of force modification factors in NBCC provide a good indication, in terms of year of construction, of the level of ductility and force capacity anticipated of a particular structural system. Thus, it is plausible to distinguish non-ductile systems from the more modern ductile systems built today in terms of year of construction in accordance with aforementioned code provisions.

Unreinforced masonry performs poorly irrespective of the year of construction. However, reinforced concrete exhibits different levels of vulnerability depending on the year of construction, which reflects the stringency of codes and standards effective during the era. Prior to 1975 there was little seismic design and detailing requirements for reinforced concrete buildings. Hence buildings designed and built in this era are viewed as having high vulnerability to seismic effects from design perspective. In 1975 the NBCC incorporated improved ductility coefficient and introduced 7 different building categories for different levels of ductility, and made reference to the 1973 edition of CSA Standard A23.3. This edition of CSA A23.3 also incorporated improved design and detaining requirements for ductile response of reinforced concrete buildings. Therefore, buildings designed after 1975 are viewed as having improved seismic resistance and ductility. Further improvements to seismic design and detailing were introduced to the 1984 edition of CSA A23.3, which was referenced by the 1985 NBCC. Hence the period between 1975 to 1985 and 1985 to 2005 when the NBCC was revised significantly, are considered to have periods of improvements in seismic design and expected seismic performance of buildings designed to conform to the corresponding codes and standards of respected periods. Current seismic design practice, representing post 2005 era have the state-of-the art design provisions and hence are expected to have minimum seismic vulnerability from design perspective. Table 3-10 provides transformation values assigned to buildings designed in different eras to reflect the effects of evolving building design and detailing practices on seismic performance of buildings. A high transformation value indicates high building vulnerability.

Table 3-10: Transformation Values for the Computation of the DDF

Building Type	**Year**	**Transformation Value**
Unreinforced Masonry	Any Year	5
Reinforced Concrete	<1975	4
	1975 to 1985	3
	1985 to 2005	2
	>2005	1

The final step in establishing the design and detailing factor (DDF), as expressed in Equation 3-4, is to multiply the spectral acceleration (at site class C) of the structural system by the transformation values to reflect different effects of this parameter in regions of different seismicity, as discussed in the previous section for seismic force factor (SFF).

Design and Detailing Factor (DDF):

$$DDF = Transformation\,Value * S_a^c(T_1) \quad [3\text{-}4]$$

Where:

- DDF is the Design and Detailing Factor
- $Transformation\,Value$ as depicted by Table 3-10
- $S_a^c(T_1)$ is the building's spectral acceleration at reference soil class C according to NBCC-2005

3.6.2 Non-Structural Vulnerability

Although the structural vulnerability of a building induces the most serious building damage, injuries and fatalities, it is evident through the performance of non-structural components of a building in recent earthquakes that the evaluation of non-structural elements must be considered in seismic risk assessment of a building (Durkin et al., 1991). In addition, a building exhibiting minor structural damage may display extensive damage to non-structural components after an earthquake. In Canada, the 1988 Saquenay earthquake caused very little structural damage but a great majority of injuries resulted from non-structural failures (Foo and Davenport, 2003). Non-structural damage in a building refers to the damage of features that do not affect the overall integrity of the structure and are not integrated as part of the structural system. This includes: architectural, mechanical, electrical, piping, plumbing, ceiling and furniture components that are included within a building (FEMA, 2010; Rainer et al., 1992). Non-structural vulnerability can be classified in three categories: i) Falling Hazards to life and ii) Damage to vital operations of strategic facilities iii) Increase in non-structural hazard. The aforementioned categories have been adopted from the Seismic Screening of Buildings in Canada tool (Saatcioglu et al., 2011) for inclusion of non-structural vulnerability assessments in a building for the CanRisk program.

3.6.2.1 Falling Hazards to Life

Falling hazards to life (FHL) include heavy non-structural components that pose a falling hazard to passers-by or areas of human occupancy. A falling hazard can be located in the interior or exterior of a building. For example, exterior falling hazards include parapets, chimneys, or cladding while interior falling hazards include operational and function components (OFCs), heavy partition walls and storage shelves (Rainer et al., 1992; Saatcioglu et al., 2011). The potential level of falling hazards to life is left to the judgement of the engineer to specify. Therefore, as define in Table 3-11, in a walk down survey the falling hazards to life are determined by five grades: very low, low, moderate, high and very high.

Table 3-11: Linguistic Input Parameters and Transformation Values for FHL

Linguistic	Transformation Value
"Very Low"	0.1
"Low"	0.3
"Moderate"	0.5
"High"	0.7
"Very High"	0.9

3.6.2.2 Hazards to Vital Operations

Hazards to vital operations (HVO) reflect seismic damage that affects the overall operational requirements of a special building and post-disaster buildings, such as a hospital. When a building is set into motion, the operability of equipment, such as medical instruments, in critical infrastructures may be compromised (FEMA, 2010; Rainer et al., 1992). The level of hazards to vital operations is left to the judgement of the engineer. Therefore, in a walk down survey, the hazards to vital operations are determined by five levels: very low, low, moderate, high and very high (see Table 3-12).

Table 3-12: Linguistic Input Parameters and Transformation Values for HVO

Linguistic	Transformation Value
"Very Low"	0.1
"Low"	0.3
"Moderate"	0.5
"High"	0.7
"Very High"	0.9

3.6.2.3 Increase in Non-Structural Hazard

Once hazard related to non-structural deficiencies of a building has been established, performance modifiers such as the year of construction and the increase in demand of a building play an important role in determining the severity of the non-structural vulnerability. For instance, the greater the seismic demand on the structure is (ID), the greater the vulnerability of the non-structural components is. In addition, the NBCC-1970 introduced horizontal force factors incorporated into the design of non-structural components for lateral forces (NRC, 1970). Prior to NBCC-1970, non-structural components were all grouped under the same category with minimal restrictions (NRC, 1965). Thus, the year of construction and the level of increase in demand of a building provide a good indication to the potential increase in non-structural hazard of a building (Saatcioglu et al., 2011).

3.6.3 Building Structural Type

The most crucial part of the seismic vulnerability assessment of a building is identifying the lateral force-resisting system designed to resist loads in the event of an earthquake (ATC, 2002). Methods for identifying the structural system in a rapid visual screening process are available in the literature, such as the guidelines outlined in Section 2.5.3 for unreinforced masonry structures and the FEMA 154 (ATC, 2002) and FEMA 310 (ASCE 1998) report for other structural systems. It is evident from reconnaissance reports and research efforts that ductile systems and shear wall buildings perform better than older brittle buildings, such as those observed to have suffered extensive damage in the 2011 Christchurch earthquake (Ingham et al., 2011). The two main types of structural systems incorporated into the modified CanRisk program developed as part of the current research program are reinforced concrete and unreinforced masonry load-bearing structural systems. The structural system is introduced at level 4 of hierarchy, illustrated in Figure 3-6 and Figure 3-7. The structural systems incorporated into the program are further described in the following sections.

3.6.3.1 Reinforced Concrete

Reinforced Concrete is a prevalent construction material because of its accessibility, low cost, strength, stiffness and ductile performance in high seismic regions (Duggal 2007). Although concrete is brittle by nature, the addition of reinforcing steel bars provide ductility to the overall structure, improving structures deformability. However, poor construction practices such as inadequate design and detailing of structural elements in older, non-ductile reinforced concrete structures have known to be much more vulnerable (Mitchell et al 1995). The three main concrete structures are defined as C1 (concrete moment frames), C2 (concrete shear wall structures) and C3 (concrete moment frames with masonry infill shear walls). The aforementioned concrete structures are defined in FEMA 310 (ASCE, 1998) and presented in Table 3-13.

Table 3-13: Concrete Building Types (ASCE, 1998)

Building Type 8: Concrete Moment Frames	
C1	These buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Lateral forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. In older construction, or in regions of low seismicity, the moment frames may consist of the column strips of two-way flat slab systems. Modern frames in regions of high seismicity have joint reinforcing, closely spaced ties, and special detailing to provide ductile performance. This detailing is not present in older construction. Foundations consist of concrete spread footings or deep pile foundations.
Building Type 9: Concrete Shear Wall Buildings	
C2	These buildings have floor and roof framing that consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Floors are supported on concrete columns or bearing walls. Lateral forces are resisted by cast-in-place concrete shear walls. In older construction, shear walls are lightly reinforced, but often extend throughout the building. In more recent construction, shear walls occur in isolated locations and are more heavily reinforced with boundary elements and closely spaced ties to provide ductile performance. The diaphragms consist of concrete slabs and are stiff relative to the walls. Foundations consist of concrete spread footings or deep pile foundations.
Building Type 10: Concrete Frames with Infill Masonry Shear Walls	
C3	This is an older type of building construction that consists of a frame assembly of cast-in-place concrete beams and columns. The floors and roof consist of cast-in-place concrete slabs. Walls consist of infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry. The seismic performance of this type of construction depends on the interaction between the frame and infill panels. The combined behavior is more like a shear wall structure than a frame structure Solidly infilled masonry panels form diagonal compression struts between the intersections of the frame members. If the walls are offset from the frame and do not fully engage the frame members, the diagonal compression struts will not develop. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The shear strength of the concrete columns, after cracking of the infill, may limit the semiductile behavior of the system. The diaphragms consist of concrete floors and are stiff relative to the walls.

Finally, the transformation values for the various reinforced concrete structural systems are presented in Table 3-14. It is important to note that for C3 buildings, the general building restrictions defined in the NBCC-2010 (Table 4.1.8.9 in NRC, 2010) was utilized to provide transformation values based on seismicity as infill frames can provide proper strength and stiffness in areas of low-to-moderate seismicity when they remain within their elastic limits, but can be damaging to the structural elements in areas of high seismicity by providing supports that have not been accounted for in design (short/captive columns) and other unintended interferences of undesirable nature.

Table 3-14: Transformation Values for Building Type of RC Structures

Structural Type	Values of Sa(0.2)	Transformation Value
C1	All	0.2
C2	All	0.4
C3	≥0.35	0.4 + (Sa(0.2)/10)
	<0.35	0.4 – ((1/Sa(0.2))/100)

3.6.3.2 Unreinforced Masonry

The original CanRisk program was developed to assess reinforced concrete structures; Part of the scope of current research is to expand the capabilities of the program to include the evaluation of unreinforced load-bearing brick/block buildings. Table 3-15 describes the URM construction type also defined in FEMA 310 (ASCE, 1998). The continued observation of URM building damage in earthquakes necessitates a critical evaluation of URM building stock in an urban centre to provide proper earthquake protection (Mitchell et al 1990). For instance, falling URM building components on passers-by and adjacent buildings caused building damage, causalities and fatalities as observed in the 1989 Loma Prieta Earthquake (Durkin et al., 1991). As the majority of URM are considered to be non-engineered, it is not surprising that URM buildings perform poorly and incur severe damage (Bruneau 1990; Durkin et al 1991; Bruneau and Lamontagne 1994). Chapter 2 provides more information on unreinforced masonry structures and the inherent seismic deficiencies of this construction type.

Table 3-15: Unreinforced Masonry Construction Type (ASCE, 1998)

Building Type 15: Unreinforced Masonry Bearing Wall Buildings	
URM	These buildings have perimeter bearing walls that consist of URM clay brick masonry. Interior bearing walls, when present, also consist of unreinforced clay brick masonry. In older construction, floor and roof framing consists of straight or diagonal lumber sheathing supported by wood joists, on posts and timbers. In more recent construction floors consist of structural panel or plywood sheathing rather than lumber sheathing. Diaphragms are flexible relative to the walls. When they exist, ties between the walls & diaphragms consist of bent steel plates or government anchors embedded in the mortar joints and attached to framing. Foundations consist of brick or concrete spread footings.

Furthermore, following the events of 2010 Darfield and the 2011 Christchurch earthquakes in New Zealand, the performance of stand-alone URM buildings were observed to have a greater degree of damage than row buildings. Similarly, whether a URM building was situated in the middle or end of a row played a major significance. URM buildings located at the end of a row typically observed more damage (referred to as 'bookend' behaviour) than a URM building in the middle of a row (Ingham and Griffith, 2011b). Thus, a set of transformation values based on the typology of a URM building are introduced in Table 3-16.

Table 3-16: Transformation Values for Building Type of URM Structures

Structural Type	Transformation Value
URM – Standalone	0.9
URM – Row (end)	0.85
URM – Row (Middle)	0.8

3.7 Building Importance/Exposure Module (I^{IE})

The level of importance of a building can be established on the basis of use and occupancy. The NBCC defines an importance factor for determining total seismic base shear based on occupancy. The NBCC classification includes normal, high importance and post-disaster buildings. Each building is assigned an importance factor (I_E) when calculating its base shear as defined in Table 3-17. High importance structures include schools and community centres that are able to house a large number of individuals. Post-disaster buildings include hospitals and emergency response facilities that are required to remain operational in the event of a disaster. Normal importance buildings include all other buildings that do not fall in the high or post-disaster categories (NRCC, 2010).

Table 3-17: Building Importance Categories as defined in NBCC-2010

Building Importance	I_E
Normal	1
High	1.3
Post-Disaster	1.5

The building importance factor is calculated using Equation 3-5, which includes both use and occupancy in order to determine the significance of the building under investigation. The formula has been adopted and modified from the Seismic Screening Tool (Saatcioglu et al., 2011).

Building Importance Factor (BIF):

$$BIF = \ Occupied\ Area\ x\ Occupancy\ Density\ x\ \frac{Avg\ Weekly\ Hours}{100}\ x\ I_E \qquad [3\text{-}5]$$

Where:

- BIF is the Building Importance Factor defined in Table 3-17
- $Occupied\ Area$ is the building's area in m^2
- $Occupancy\ Density$ is the estimated average number of people occupied per m^2
- $Avg\ Weekly\ Hours$ is the average number of hours weekly occupied in the building
- I_E is the building importance factor as defined in NBCC-2010 (NRC, 2010)

3.8 Building Damageability Index (I^{BD})

The building damageability index integrates the site seismic hazard and the building's vulnerability to establish damage potential for structural and non-structural components of a building. The structural building damageability index (I_S^{BD}) and the non-structural building damageability (I_{NS}^{BD}) are represented at level 2 of Figure 3-6, and Figure 3-7 respectively. As defined in Table 3-18, the damage levels of CanRisk have 5 gradations in comparison with ATC-13 damage states, which have 7 gradations.

Table 3-18: Comparison of Building Damage States of ATC-13 and CanRisk

Damage State (ATC-13)	Damage Range Factor (%)	Damage Levels (CanRisk)	Damageability Index Range (I^{BD})	Description
None	0	None	0.0 – 0.2	No damage.
Slight	0-1			Limited localized minor not requiring repair.
Light	1-10	Light	0.2 – 0.4	Significant localized damage of some components generally not requiring repair.
Moderate	10-30	Moderate	0.4 – 0.6	Significant localized damage of many components warranting repair.
Heavy	30-60	Heavy	0.6 – 0.8	Extensive damage requiring major repairs.
Major	60-100	At/Near Collapse	0.8 – 1.0	Major widespread damage that may result in the facility being razed, demolished, or repaired.
Destroyed	100			Total destruction of the majority of the facility.

3.9 Risk Index (I^R)

The final step in the hierarchical evaluation includes evaluating the risk index (I^R). The risk index is computed by integrating the building damageability index (I^{BD}) and the building importance/exposure index (I^{IE}). This step is completed at level 1 (see Figure 3-6 and Figure 3-7) to determine the structural risk index (I_S^R) and non-structural risk index (I_{NS}^R) for a given building. Subsequently, an overall building risk index (I_{ALL}^R) is established by combining the structural risk and non-structural risk for the building under investigation. The risk is divided into four levels of gradation. The corresponding range of indices is defined in Table 3-19.

Table 3-19: CanRisk Risk Level and Risk Index Range

Risk Level	Risk Index Range (I^R)
Negligible	0.0 – 0.2
Marginal	0.2 – 0.4
Critical	0.4 – 0.6
Catastrophic Consequences	0.6 – 1.0

3.10 Fuzzification Summary

Table 3-20: Summary of fuzzification of hierarchical structures

Input	Transformation	Fuzzification	FRB	Output
VI	{VL, L, M, H, VH} {0.1, 0.3, 0.5, 0.7, 0.9}	{VL, L, M, H, VH} [TFN(-0.25,0,0.15); TFN(-0.1,0.15,0.4); TFN(0.15,0.4,0.65); TFN(0.4,0.65,0.9); TFN(0.65,0.9,100)]	R1	ID
PI	{VL, L, M, H, VH} {0.1, 0.3, 0.5, 0.7, 0.9}	{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]		
YOC - SFF	Eq. [3-3	{VL, L, M, H, VH} [TFN(0,0,1); TFN(0,1,2); TFN(1,2.5,4); TFN(3,4,5); TFN(4,5,1000)]	R2A	YOC
YOC - DDF	Eq. [3-4	{VL, L, M, H, VH} [TFN(0,0,1); TFN(0,1,2); TFN(1,2.5,4); TFN(3,4,5); TFN(4,5,1000)]		
YOC		{VL, L, M, H, VH} [TFN(-0.25,0,0.15); TFN(-0.1,0.15,0.4); TFN(0.15,0.4,0.65); TFN(0.4,0.65,0.9); TFN(0.65,0.9,100)]	R2B	DR
CQ	{Good, Average, Poor} {0.1, 0.5, 0.9}	{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]		
ID		{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]	R3	SV
DR		{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]		
SV		{VL, L, M, H, VH} [TFN(-0.25,0,0.15); TFN(-0.1,0.15,0.4); TFN(0.15,0.4,0.65); TFN(0.4,0.65,0.9); TFN(0.65,0.9,100)]	R4	SBV
BSS	Table 3-14	{VL,L, M, H, VH} [TFN(-0.25,0,0.15); TFN(-0.1,0.15,0.4); TFN(0.15,0.4,0.65); TFN(0.4,0.65,0.9); TFN(0.65,0.9,100)]		
SBV		{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]	R5	SBD
SSH	Sa(T1)	{VL, L, M, H, VH} [TFN(-0.4375,0,0.3); TFN(0,0.3,0.5); TFN(0.3,0.5,0.7); TFN(0.5,0.7,1.1); TFN(0.7,1,1000)]		

Input	Transformation	Fuzzification	FRB	Output
EI	{Negligible, Average, Significant} {0.1, 0.5, 0.9}	{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]	R6	BI/E
BU	Eq. [3-5	{Low, Normal, High, Post Disaster} [TPFN(-900,-100,5,50); TPFN(5,50,500,750); TPFN(500,750,2700,3000); TPFN(2700,3000,9000,100000)]		
SBD		{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]	R7	SR
BI/E		{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]		
YOC	Year Selected	{L,H} [TPFN(1910,1920,1969,1971); TPFN(1969,1971,2010,2010)]	R8	INSH
ID		{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]		
FHL	{VL, L, M, H, VH} {0.1, 0.3, 0.5, 0.7, 0.9}	{VL, L, M, H, VH} [TFN(-0.25,0,0.15); TFN(-0.1,0.15,0.4); TFN(0.15,0.4,0.65); TFN(0.4,0.65,0.9); TFN(0.65,0.9,100)]	R9	NSD
HVO	{VL, L, M, H, VH} {0.1, 0.3, 0.5, 0.7, 0.9}	{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]		
INSH		{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]	R10	NSV
NSD		{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]		
NSV		{VL, L, M, H, VH} [TFN(-0.25,0,0.15); TFN(-0.1,0.15,0.4); TFN(0.15,0.4,0.65); TFN(0.4,0.65,0.9); TFN(0.65,0.9,100)]	R11	NSBV
BSS	Table 3-14	{VL, L, M, H, VH} [TFN(-0.25,0,0.15); TFN(-0.1,0.15,0.4); TFN(0.15,0.4,0.65); TFN(0.4,0.65,0.9); TFN(0.65,0.9,100)]		

Input	Transformation	Fuzzification	FRB	Output
NSBD		{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]	R12	NSBD
SSH	Sa(T1)	{VL, L, M, H, VH} [TFN(-0.4375,0,0.3); TFN(0,0.3,0.5); TFN(0.3,0.5,0.7); TFN(0.5,0.7,1.1); TFN(0.7,1,1000)]		
NSBD		{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]	R13	NSR
BI/E		{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]		
SR		{VL, L, M, H, VH} [TFN(-0.25,0,0.15); TFN(-0.1,0.15,0.4); TFN(0.15,0.4,0.65); TFN(0.4,0.65,0.9); TFN(0.65,0.9,100)]	R14	OBR
NSR		{VL, L, M, H, VH} [TFN(-0.25,0,0.25); TFN(0,0.25,0.5); TFN(0.25,0.5,0.75); TFN(0.5,0.75,1.0); TFN(0.75,1,1.25)]		

Chapter 4
Sensitivity Analysis and Model Verification

4.1 General

This chapter provides information related to the testing and verification of CanRisk. A sensitivity analysis is conducted to assess the significance of selected parameters on seismic risk. In addition, the observed performance of URM buildings in the 2011 Christchurch earthquake in New Zealand was used to verify the method used in CanRisk.

4.2 Sensitivity Analysis

The sensitivity of input parameters provides an understanding of the significance of each parameter on the result. This provides verification of trends against those expected based on knowledge, experience and available data. For example, as the increase in demand and decrease in resistance of a building are high, the resulting change in a building's structural vulnerability should be high. As well, the effects of certain parameters are known to be more pronounced than others, and this can be checked against the results of the sensitivity analysis. This section provides a summary of the analysis conducted to evaluate the sensitivity of parameters in CanRisk.

4.2.1 Independent Input Parameters

As part of the sensitivity analysis, the aggregation of input parameters (performance modifiers) using the rule base expressions (R_i) represented in Figure 3-6 and Figure 3-7 are tested. This is achieved by varying the optional input parameters to observe their effects on the resulting output parameters.

For example, as represented in the structural risk hierarchy at level 6 in Figure 3-6, *vertical irregularity* (VI) and *plan irregularity* (PI) are independent input parameters which are combined using fuzzy rule base 1 (R1) in the hierarchical structure to provide insight on the potential *increase in demand* (ID) of a building. Figure 4-1 illustrates the possible outcomes of the combination of VI and PI input parameters for ID of a building. It is clear that as the *vertical irregularity* and *plan irregularity* of a building increase so does the *increase in* *demand* of the building. As the evaluation process continues, the resulting output value for ID is used as an input at the next level of the hierarchy (i.e. when *increase in demand* is aggregated with *decrease in resistance* to give *structural vulnerability*). The *construction quality* (CQ) and *year of construction* (YOC) are aggregated using fuzzy rule base 3 (R3) to provide the *decrease in resistance* (DR) of the structure (see level 5 of Figure 3-6). Figure 4-2 shows the results of a building's *structural vulnerability* (level 4 of Figure 3-6) as a variation of the input irregularities (for the increase in *demand*) of a building built prior to 1940 with poor CQ. The results show possible output values for *structural vulnerability* for the most severe circumstances of *decrease in resistance* of a building. In contrast, Figure 4-3 represents the results for structural vulnerability in terms of the best case circumstances for DR of a building. This is considered to be a building built according to modern codes with good construction quality. It is clear that a building with poor *construction quality* built prior to 1940 results in a higher *structural vulnerability* value in comparison to a building built in 2010 with good *construction quality*.

The input parameters are also tested to provide results for non-structural risk hierarchy (Figure 3-7). The *falling hazards to life* (FHL) and *hazards to vital operations* (HVO) are aggregated using fuzzy rule base 9 (R9) to compute the *non-structural deficiency* (NSD) of a building (see level 5 of Figure 3-7). Figure 4-4 provides the results for the *non-structural deficiency* of a building as FHL and HVO input parameters vary. It is evident that as the level of FHL and HVO increase, so does the NSD of a building. Once the value for NSD is obtained, the resulting output value is used as input for the next level of hierarchy (i.e. when *non-structural deficiency* is aggregated with *increase in non-structural hazard* to compute *non-structural vulnerability*). The increase in non-structural hazard (INSH) is a function of the *year of construction* and the level of *increase in demand* of a building. Figure 4-5 represents the non-structural vulnerability (NSV) results for the variation of non-structural hazards for the most severe circumstances of *increase in non-structural hazard* (built before 1970 with a very high ID). In contrast, Figure 4-6, shows the results for the NSV of a building in terms of the best case circumstances of the INSH of a building. This would be a building built according to the modern code with a very low ID. It is clear that buildings built prior to

1970 with very high *increase in demand* result in larger *non-structural vulnerability* values when compared with buildings built in 2010 with very low *increase in demand*.

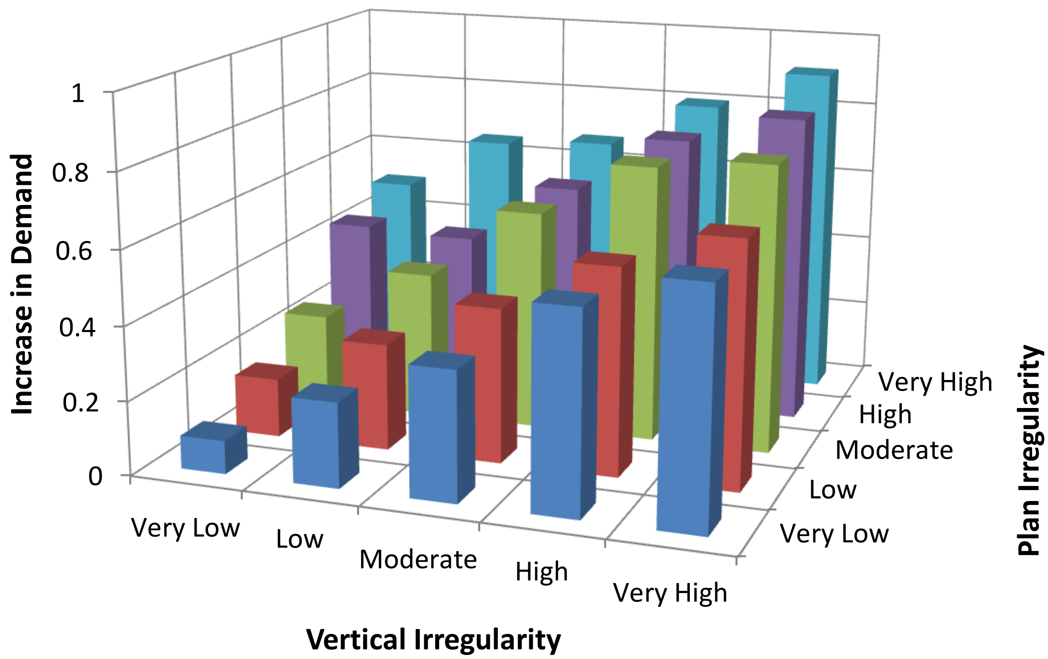


Figure 4-1: Increase in Demand of a Building

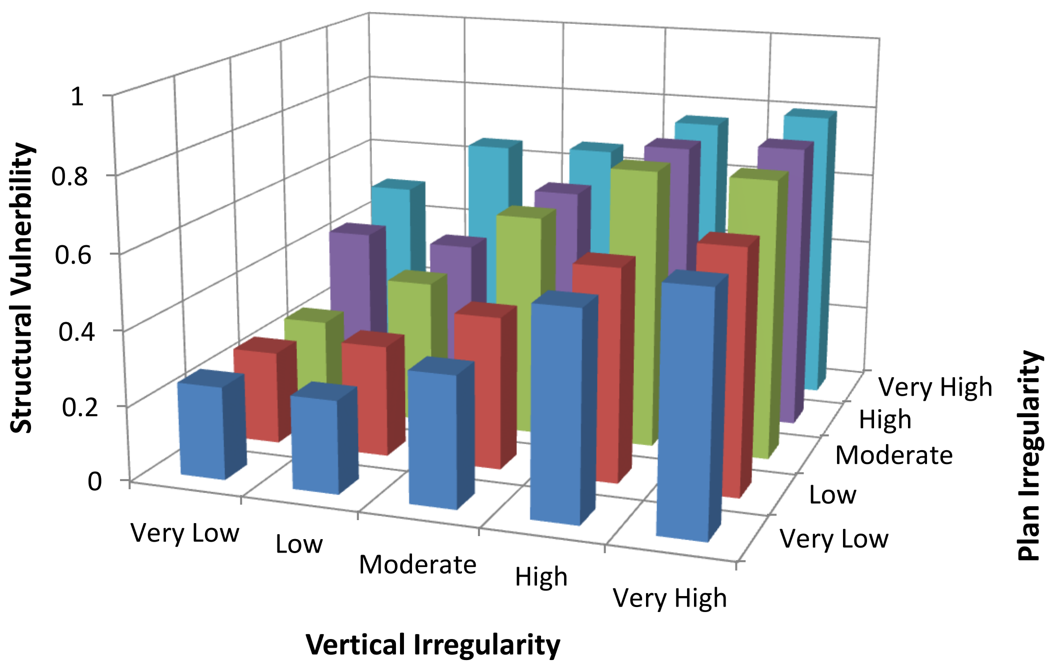


Figure 4-2: Structural Vulnerability of a Building built prior to 1940 with poor construction quality

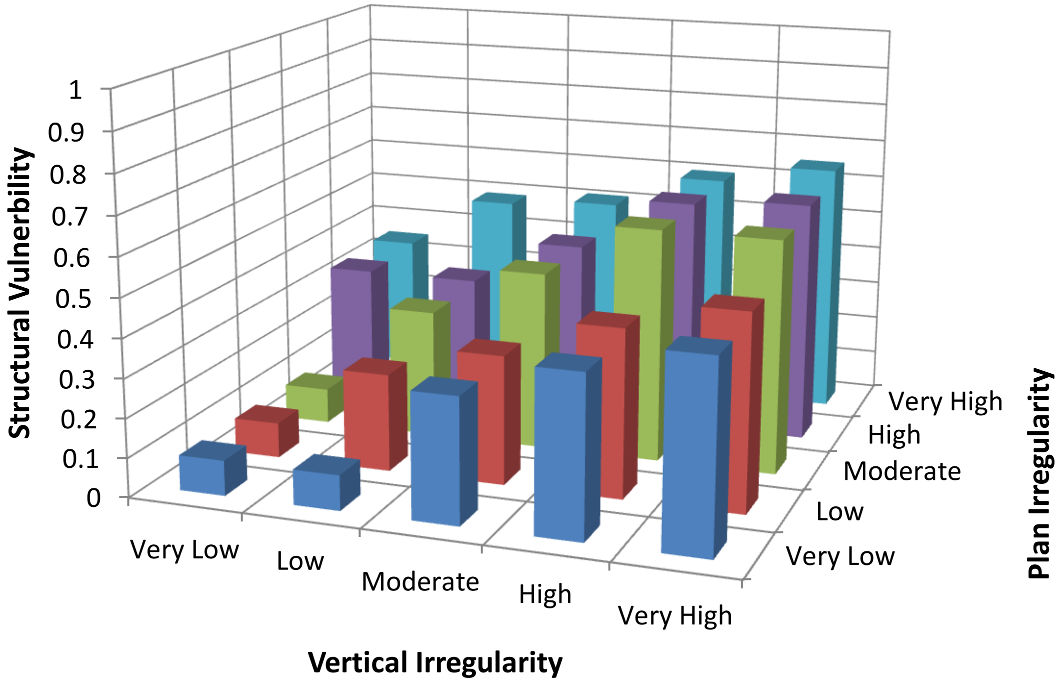


Figure 4-3: Structural Vulnerability of a Building built in 2010 with good construction quality

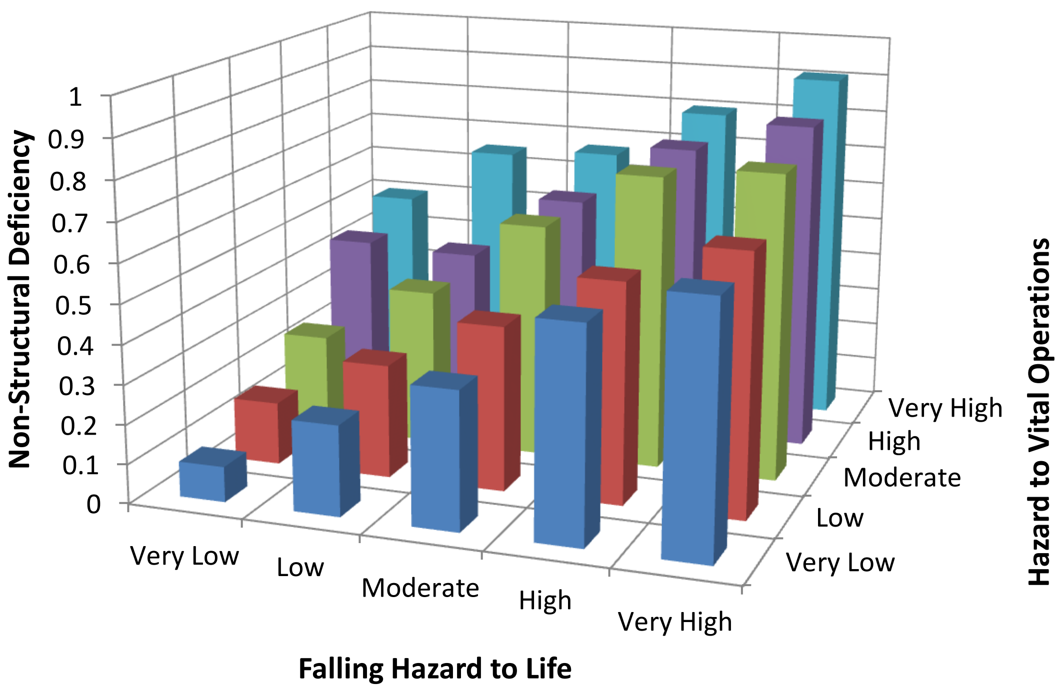


Figure 4-4: Non-structural deficiency of a Building

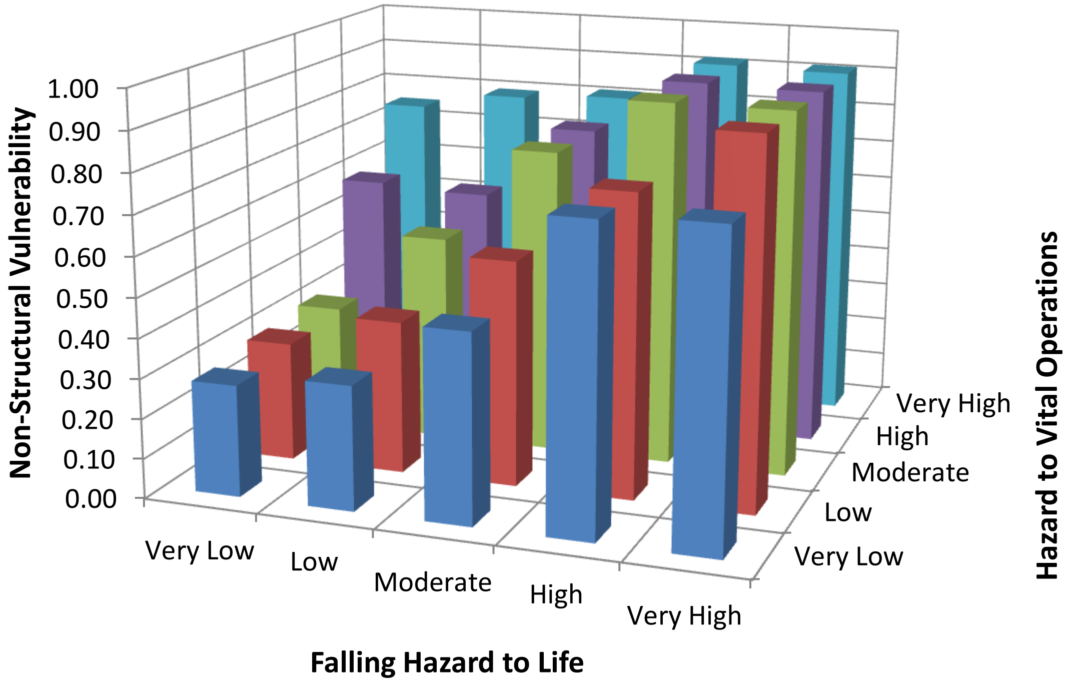


Figure 4-5: Non-Structural Vulnerability of a Building built prior to 1970 with very high ID

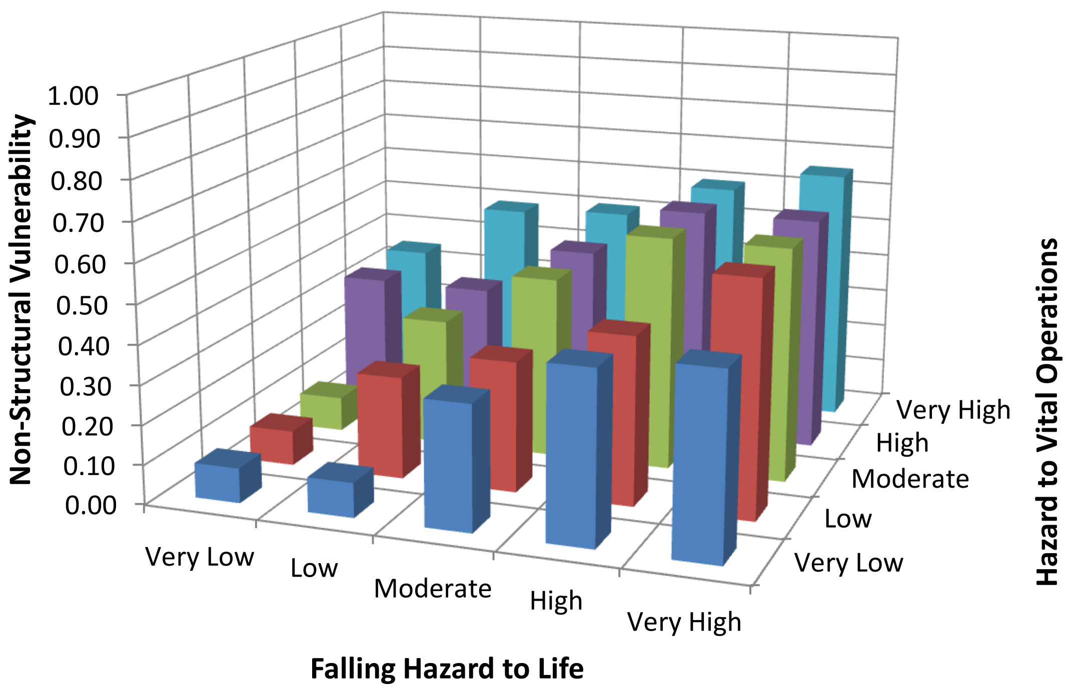


Figure 4-6: Non-Structural Vulnerability of a Building built in 2010 with very low ID

4.2.2 Structural Building Damageability Index

In addition to testing the sensitivity of individual input parameters, the overall impact of the aggregation of multiple parameters and modules is tested. In this section the sensitivity of *structural building damageability index* (I_S^{BD}) is presented (see level 2 of Figure 3-6).

The *structural building damageability index* (I_S^{BD}) is computed by varying input parameters in the structural risk hierarchy to obtain upper and lower bound limits for I_S^{BD}. To test sensitivity, the *structural building vulnerability* (SBV) evaluated at extreme case scenarios (very low and very high vulnerability) are aggregated with varying levels of *site seismic hazard* (SSH) by considering a range of possible soil conditions and number of stories (building period).

Figure 4-7 to Figure 4-22 show the results of this analysis for building types C1, C2, C3 and URM located in Edmonton, Ottawa, and Victoria, which represent areas of low, moderate and high seismicity, respectively. In these figures, the upper bound limits (dotted curves) represent the performance of a building evaluated for worst case circumstances (very high *structural building vulnerability*), while lower bound limits (solid curves) represent the performance of a building evaluated at best case circumstances (very low *structural building vulnerability*). It is noted that the results are shown for varying soil conditions and the results are plotted as a function of building storey (which influences building period).

Examination of the variations and trends in the figures for *structural building damageability index* (I_S^{BD}) shows that buildings in the short period range are experiencing higher damage levels (as reflected in higher I_S^{BD} values) when compared with longer period structures. This conforms to the modern seismic design force requirements, where the general trend is for forces to increase in the short period region and decrease in the long-period region (Heidebrecht, 2003).

Considering Figure 4-7 to Figure 4-18, the results demonstrate that higher damage levels are observed in softer soil profiles (site class D & E) in the short to moderate period range, which follows the expected trend. Furthermore, as seismicity increases so does the range of damageability between the upper and lower bound limits (the lower and upper bound curves are further apart). For example, for building type C1, Figure 4-9 shows a large range of building damageability when considering upper and lower bound limits in Victoria (high seismicity), whereas as shown in Figure 4-7 the same building type shows a smaller range of values for *structural building damageability* in Edmonton (low seismicity). Furthermore, as shown in Figure 4-19 to Figure 4-22, as seismicity increases so does *structural building damageability*, which is expected (the curves are shown for site class C).

The results also show high vulnerability of URM buildings, which show relatively higher damageability values when compared with building types C1, C2, C3 (Figure 4-23 to Figure 4-25). In addition, the variation in damageability is reduced for this building type (lower and upper bound curves are closer together) when compared with other building types. Furthermore, Figure 4-26 demonstrates that row URM buildings perform better than stand-alone URM buildings (for Ottawa, Ontario). Thus, when *structural vulnerability* is very low (solid curves), URM row buildings (irrespective of position, i.e. row-end or row-middle) perform better than URM stand-alone buildings. When *structural vulnerability* is very high (dotted curves), row-middle URM performs better than row-end URM, which performs better than URM stand-alone buildings.

The results for concrete buildings generally demonstrate overall reduced damageability for shear wall buildings (C2) when compared with concrete moment frame buildings (C1 and C3). This is observed in Figure 4-20 (C2 buildings) in comparison with Figure 4-19 and Figure 4-21 (C1 and C3 buildings), for buildings located in different areas of seismicity. For concrete moment frame buildings, buildings with masonry infill shear walls perform well in regions of lower seismicity due to the bracing provided by the infill, whereas the contrary is true for buildings in high-seismicity regions due to the negative effect of infills on structural moment frames beyond the elastic limit of URM. *Structural building damageability* for C3 structures increases as the seismicity of a region increases. This is shown in Figure 4-24 for Ottawa, ON, compared with Figure 4-25 for Victoria, BC. For concrete moment frame buildings without masonry infill shear walls (C1), the results show that this structure type may perform better than C2 and C3 because of the higher forces attracted by shear wall buildings.

Overall, the sensitivity of *structural building damageability index* (I_S^{BD}) illustrates the poor performance of buildings on softer soil profiles, in comparison with buildings on hard rock soil profiles. In addition, C1 and C2 buildings perform generally the best, followed by C3 buildings. It is evident that URM buildings perform the poorest among the structural types considered. Finally, URM stand-alone buildings perform generally poor in comparison to URM buildings in a row configuration.

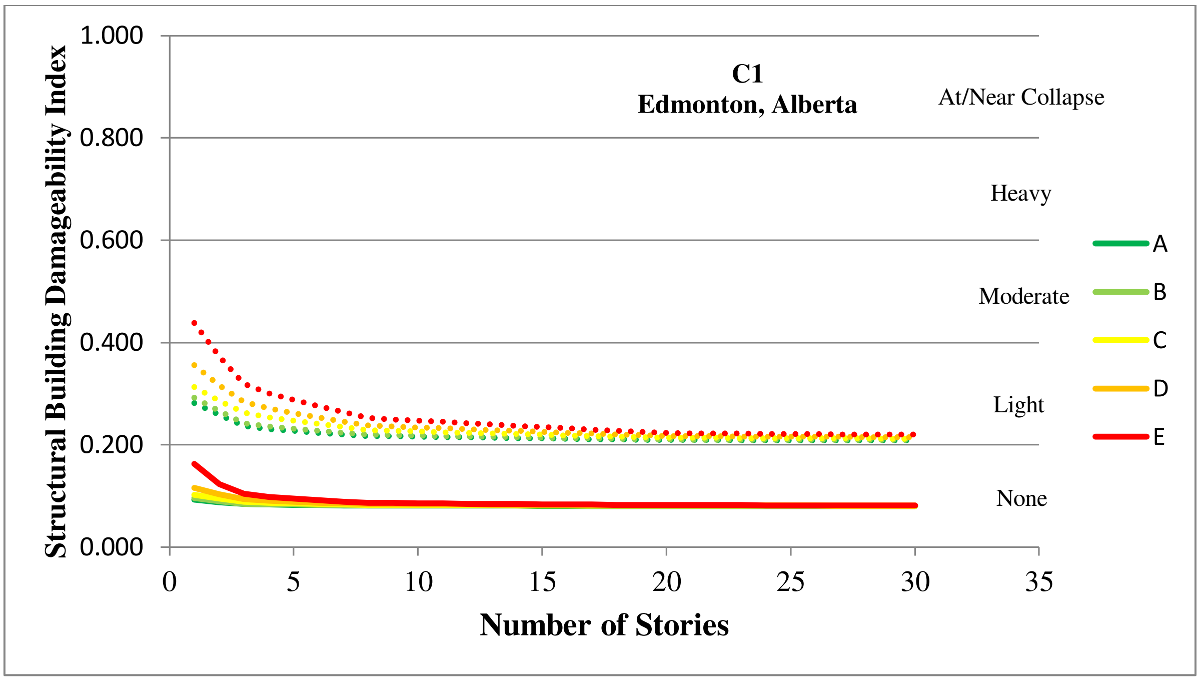


Figure 4-7: Upper and lower bound limits of I^{BD} for C1 structures resting on various soil conditions in Edmonton, Alberta

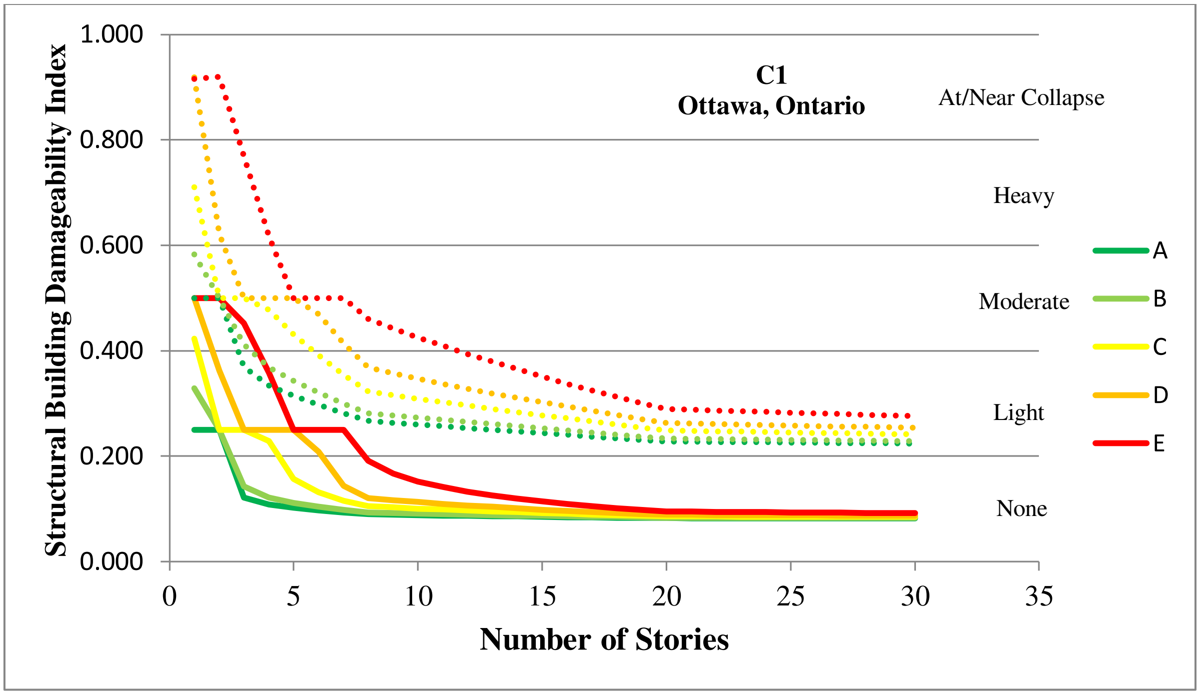


Figure 4-8: Upper and Lower Bound Limits of Building Damageability for C1 Structures resting on various soil conditions in Ottawa, Ontario

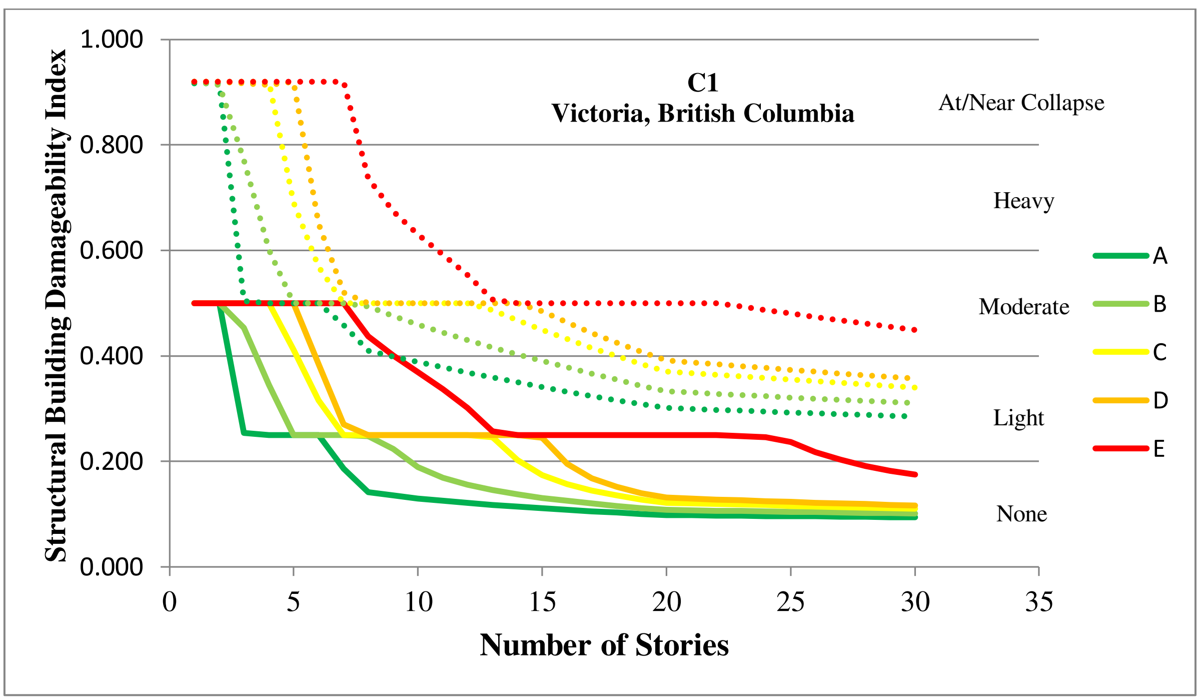


Figure 4-9: Upper and Lower Bound Limits of Building Damageability for C1 Structures resting on various soil conditions in Victoria, British Columbia

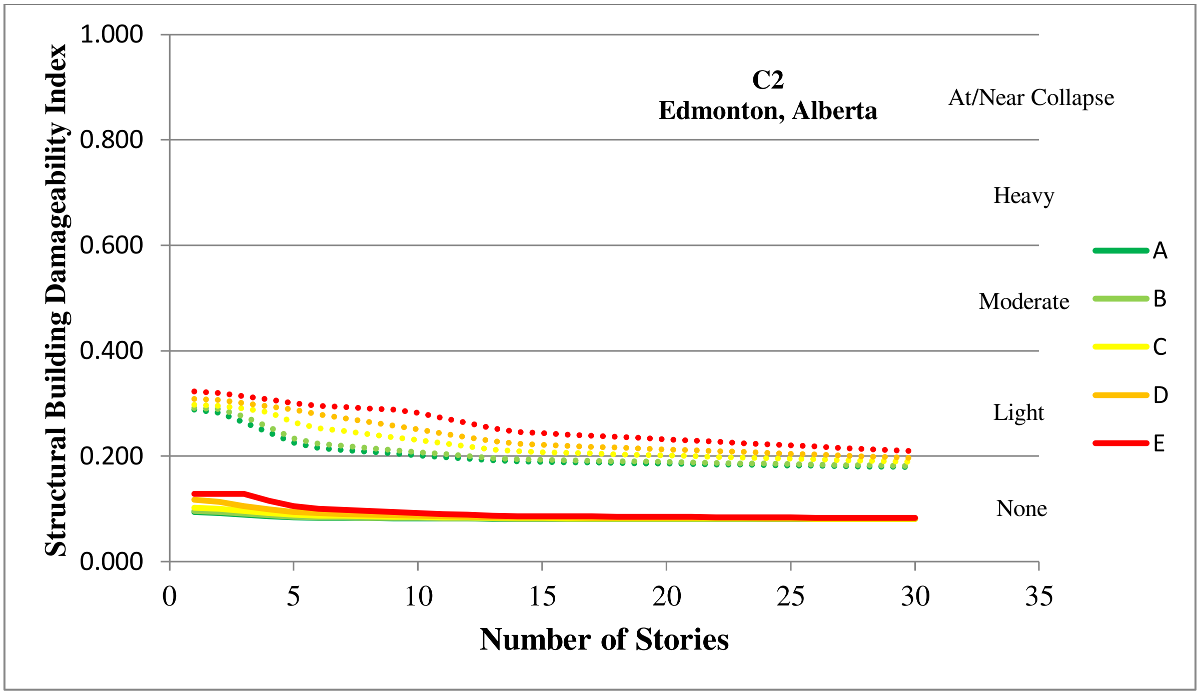


Figure 4-10: Upper and Lower Bound Limits of Building Damageability for C2 Structures resting on various soil conditions in Edmonton, Alberta

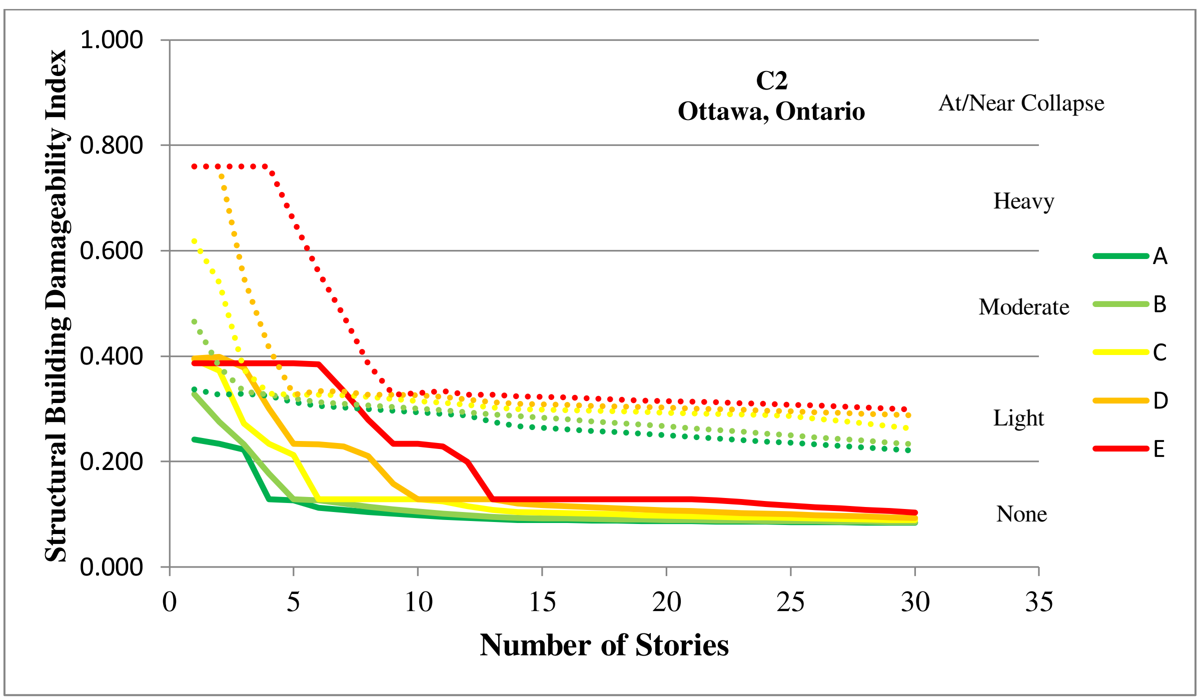


Figure 4-11: Upper and Lower Bound Limits of Building Damageability for C2 Structures resting on various soil conditions in Ottawa, Ontario

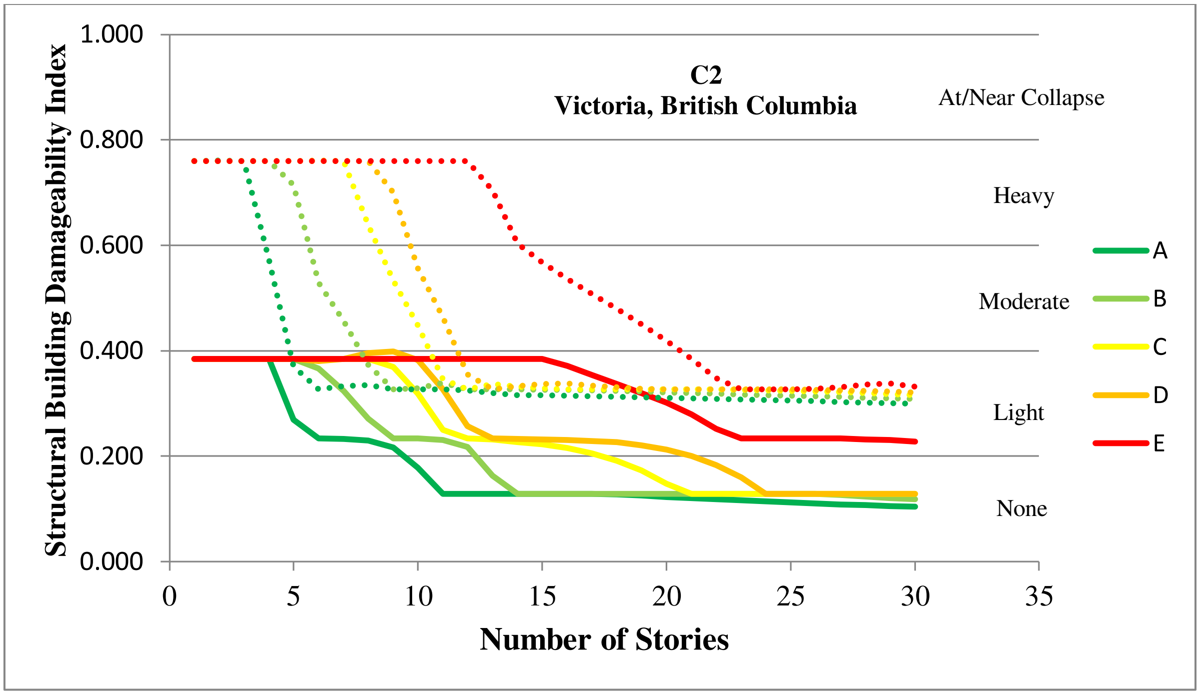


Figure 4-12: Upper and Lower Bound Limits of Building Damageability for C2 Structures resting on various soil conditions in Victoria, British Columbia

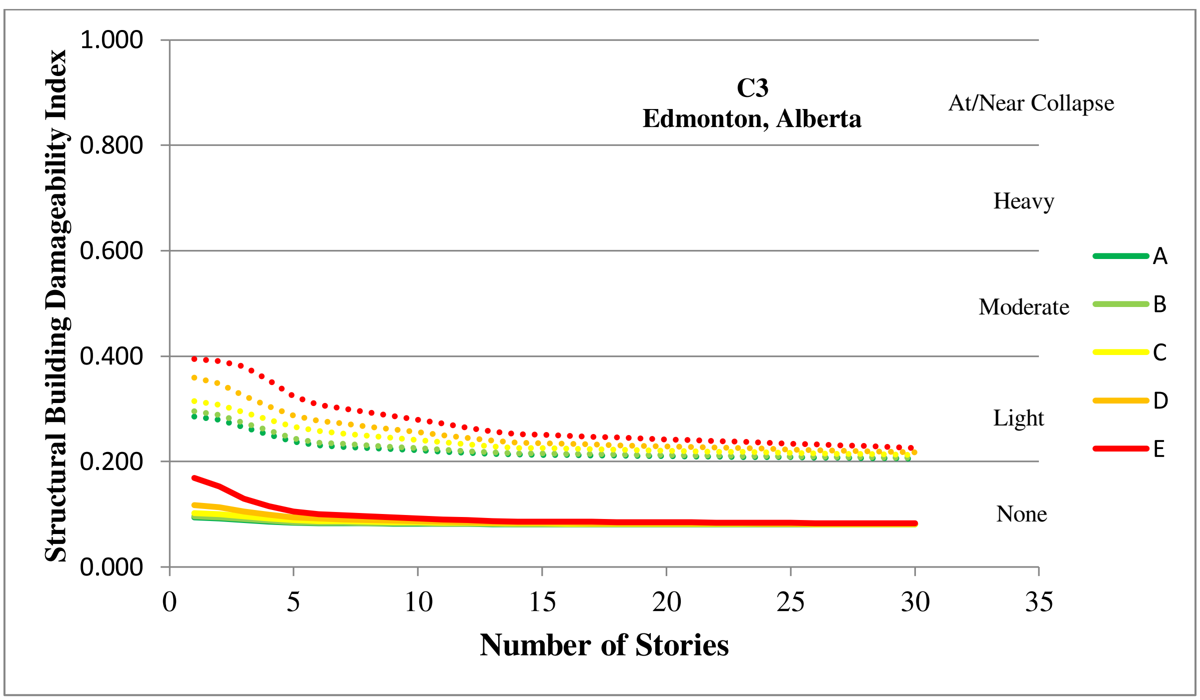


Figure 4-13: Upper and Lower Bound Limits of Building Damageability for C3 Structures resting on various soil conditions in Edmonton, Alberta

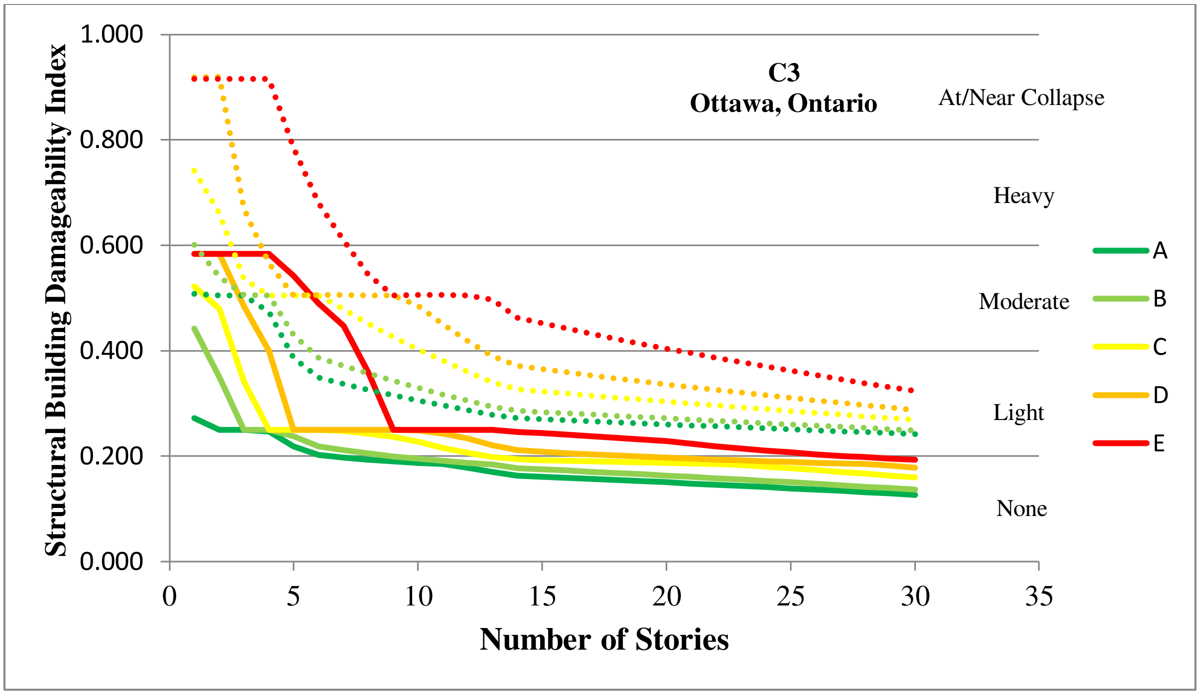


Figure 4-14: Upper and Lower Bound Limits of Building Damageability for C3 Structures resting on various soil conditions in Ottawa, Ontario

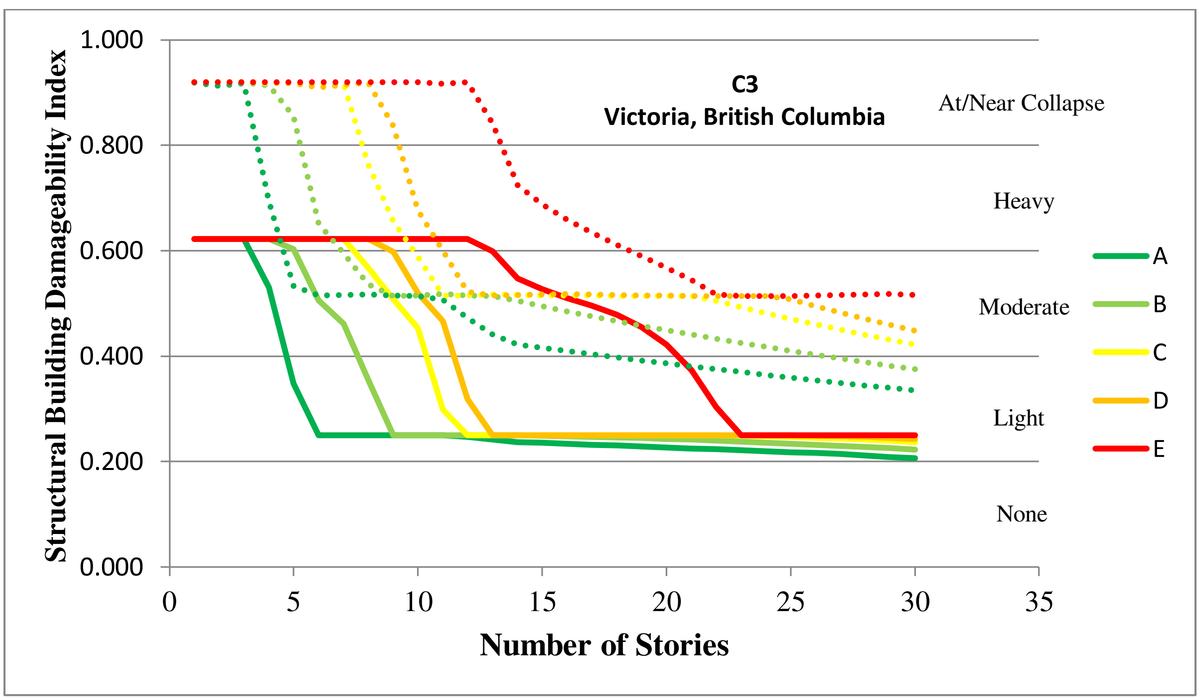


Figure 4-15: Upper and Lower Bound Limits of Building Damageability for C3 Structures resting on various soil conditions in Victoria, British Columbia

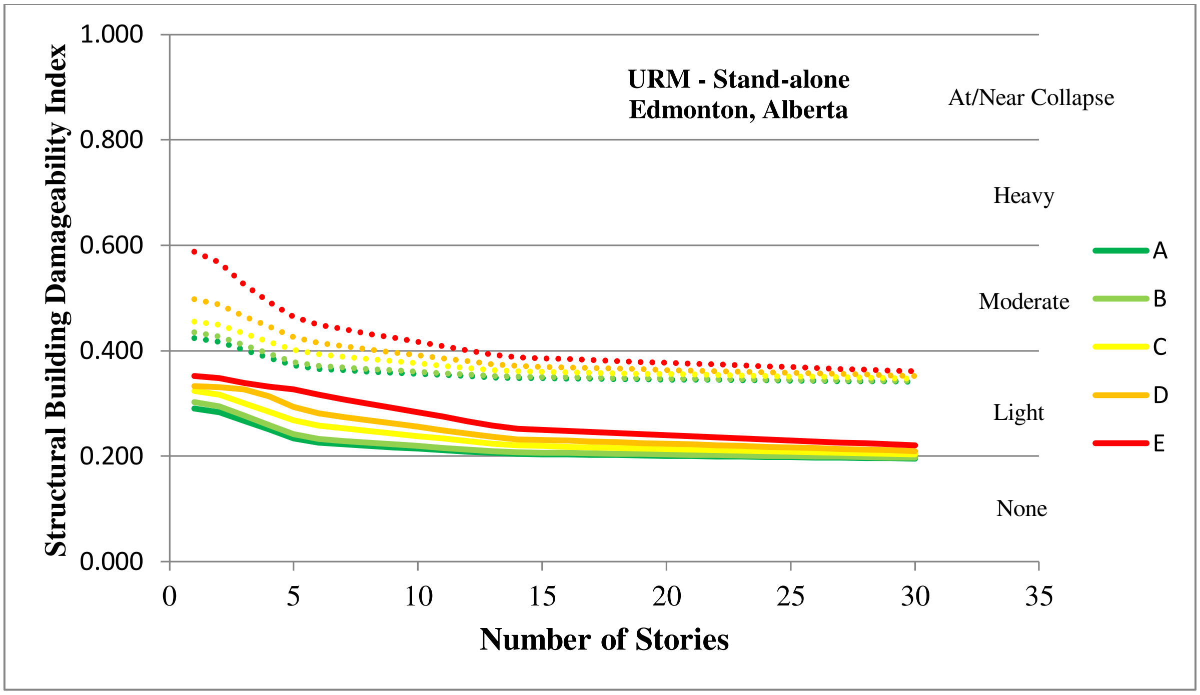


Figure 4-16: Upper and Lower Bound Limits of Building Damageability for URM (Stand-alone) Structures resting on various soil conditions in Edmonton, Alberta

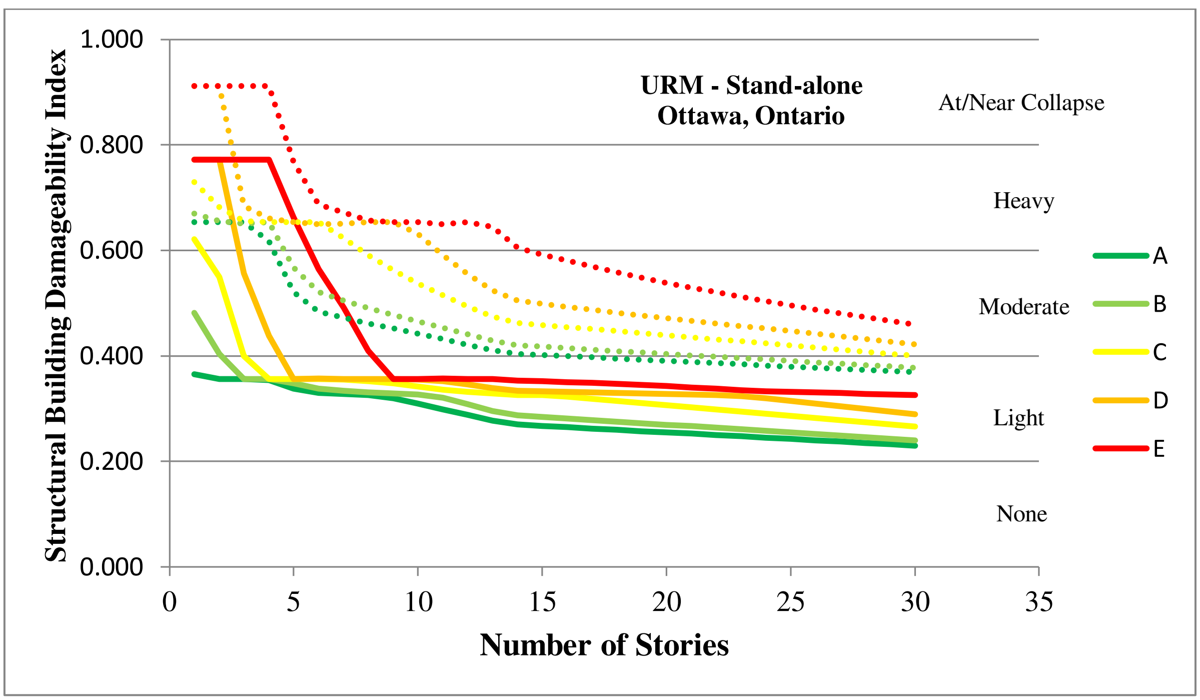


Figure 4-17: Upper and Lower Bound Limits of Building Damageability for URM Structures resting on various soil conditions in Ottawa, Ontario

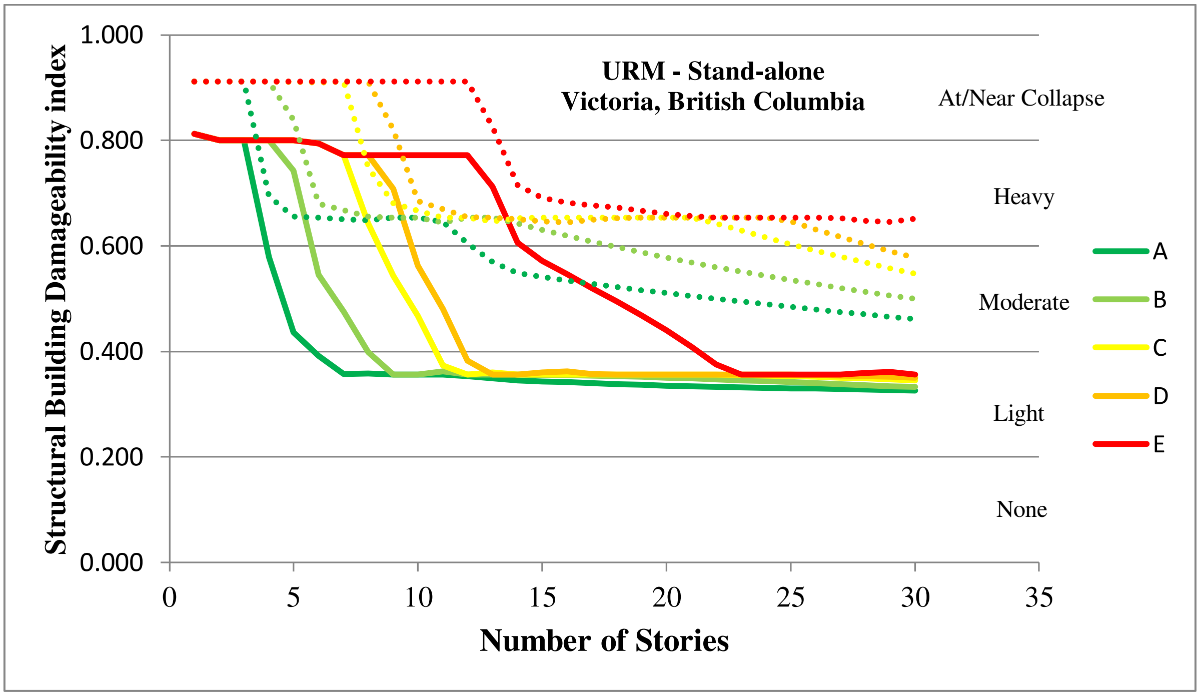


Figure 4-18: Upper and Lower Bound Limits of Building Damageability for URM Structures resting on various soil conditions in Victoria, British Columbia

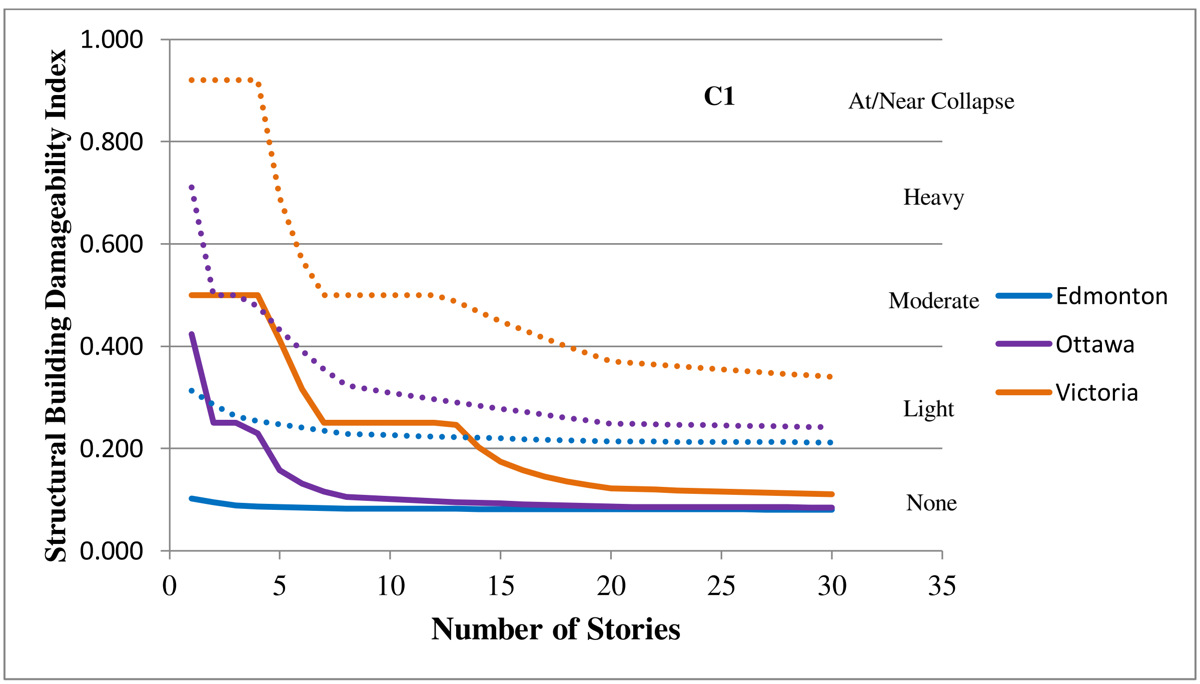


Figure 4-19: Comparison of Upper and Lower Bound Limits of Building Damageability for C1 Structures Resting on Reference Site Soil Class C

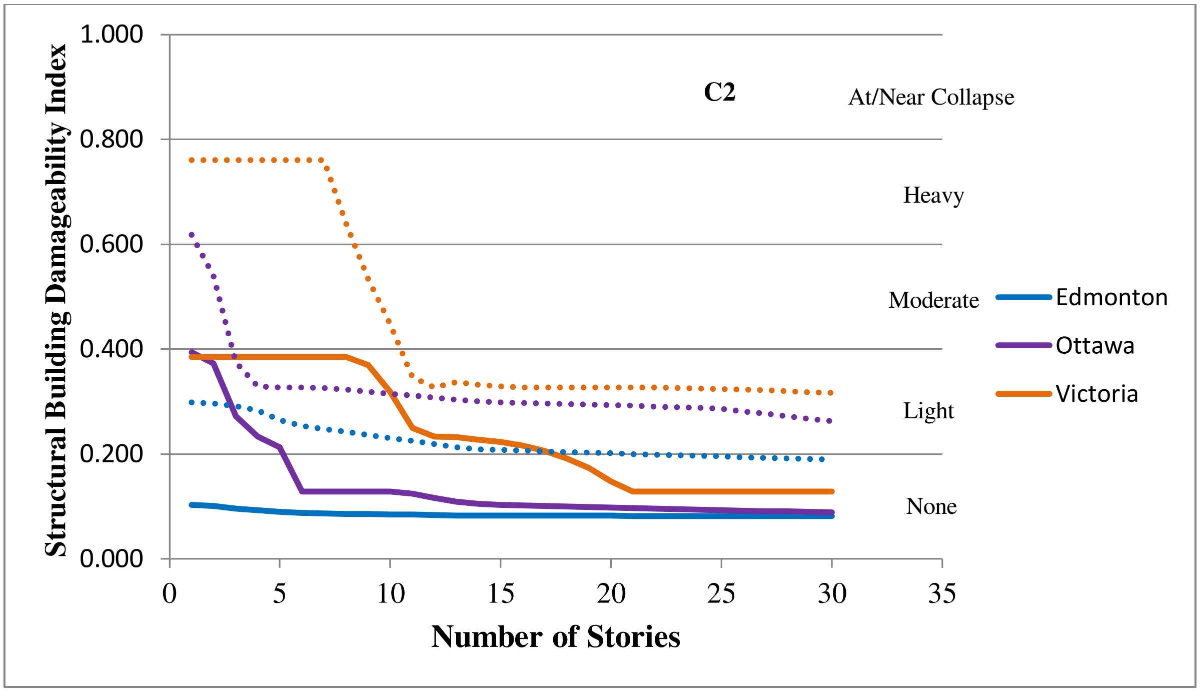


Figure 4-20: Comparison of Upper and Lower Bound Limits of Building Damageability for C2 Structures Resting on Reference Site Soil Class C

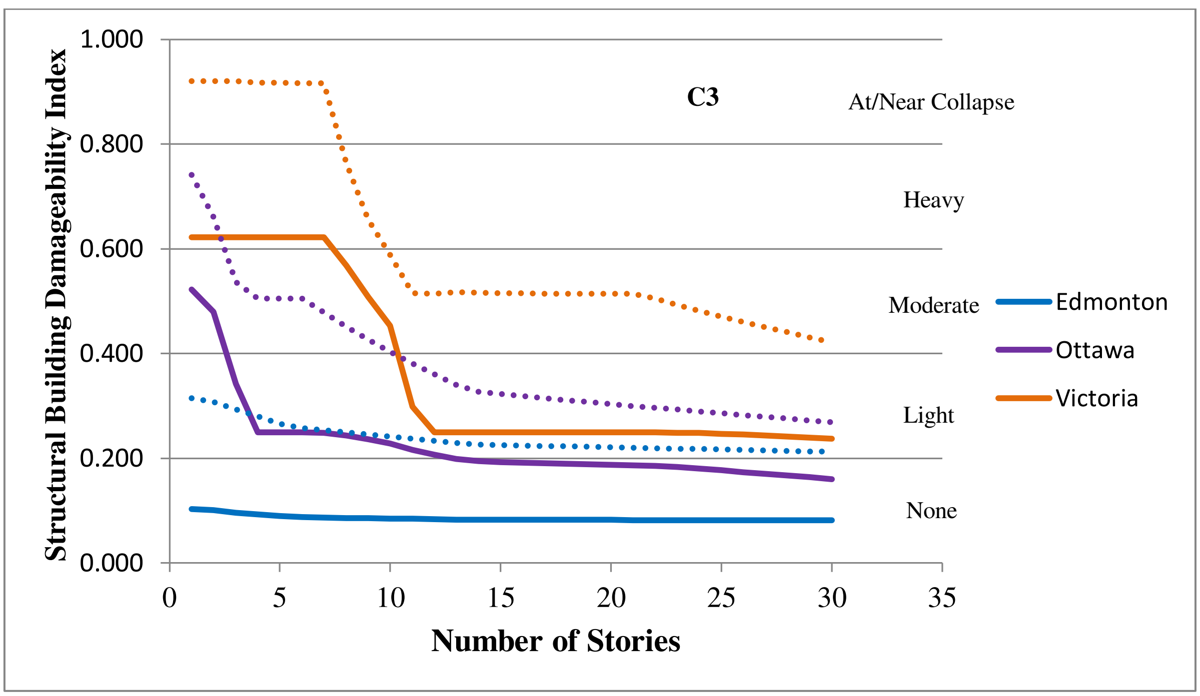


Figure 4-21: Comparison of Upper and Lower Bound Limits of Building Damageability for C3 Structures Resting on Reference Site Soil Class C

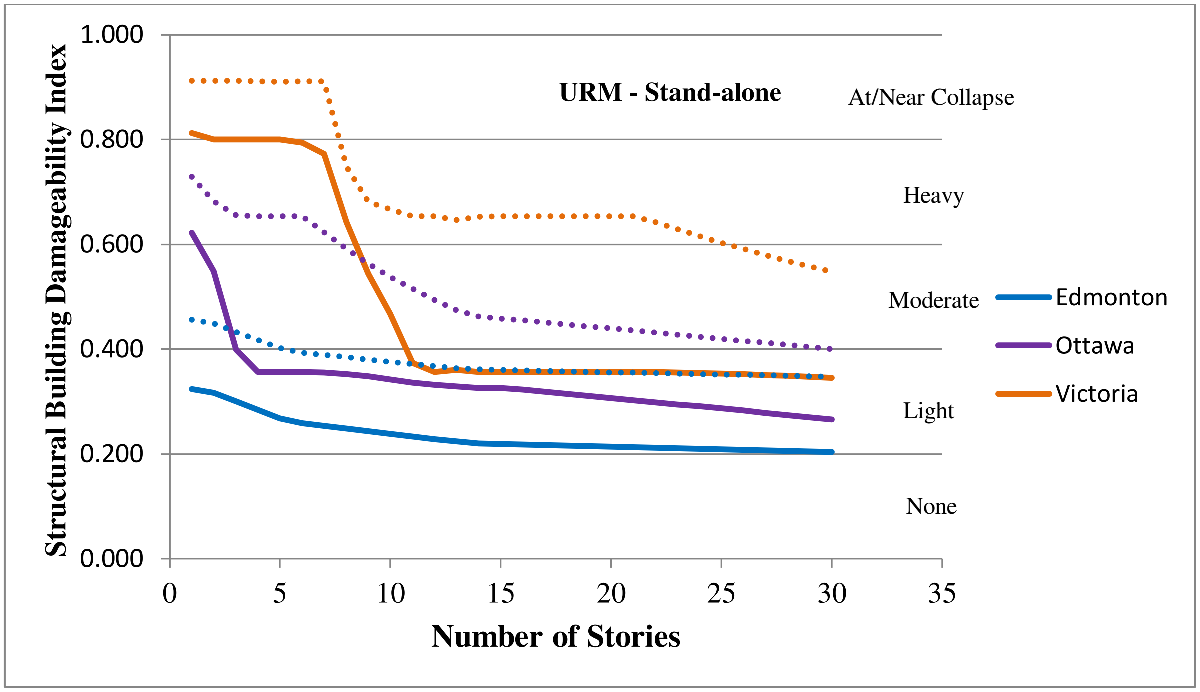


Figure 4-22: Comparison of Upper and Lower Bound Limits of Building Damageability for URM Structures Resting on Reference Site Soil Class C

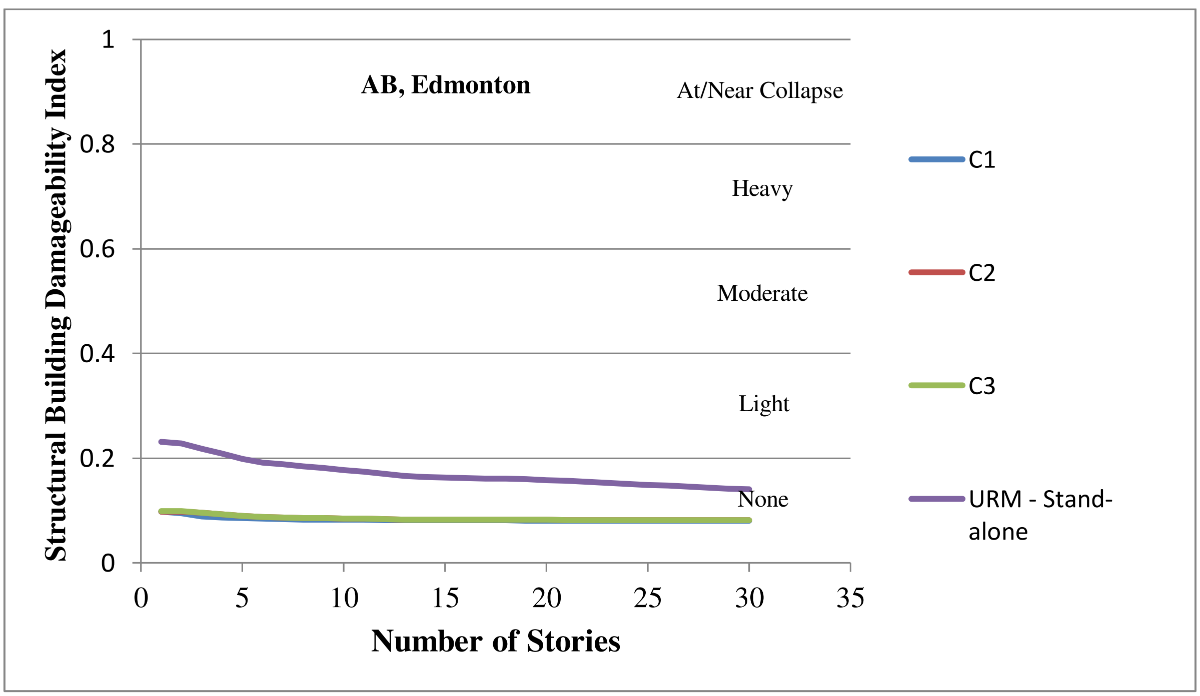


Figure 4-23: Comparison of Building Type for Structural Building Damageability of Structures Resting on Reference Site Soil Class C in Edmonton, AB

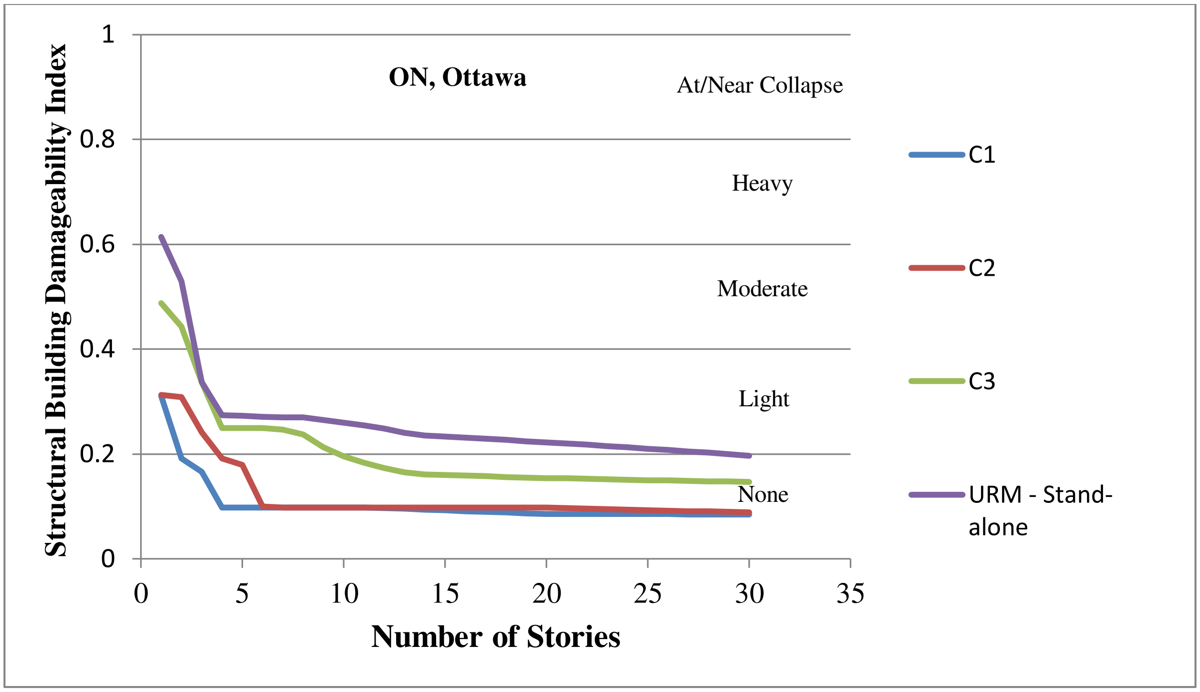


Figure 4-24: Comparison of Building Type for Structural Building Damageability of Structures Resting on Reference Site Soil Class C in Ottawa, ON

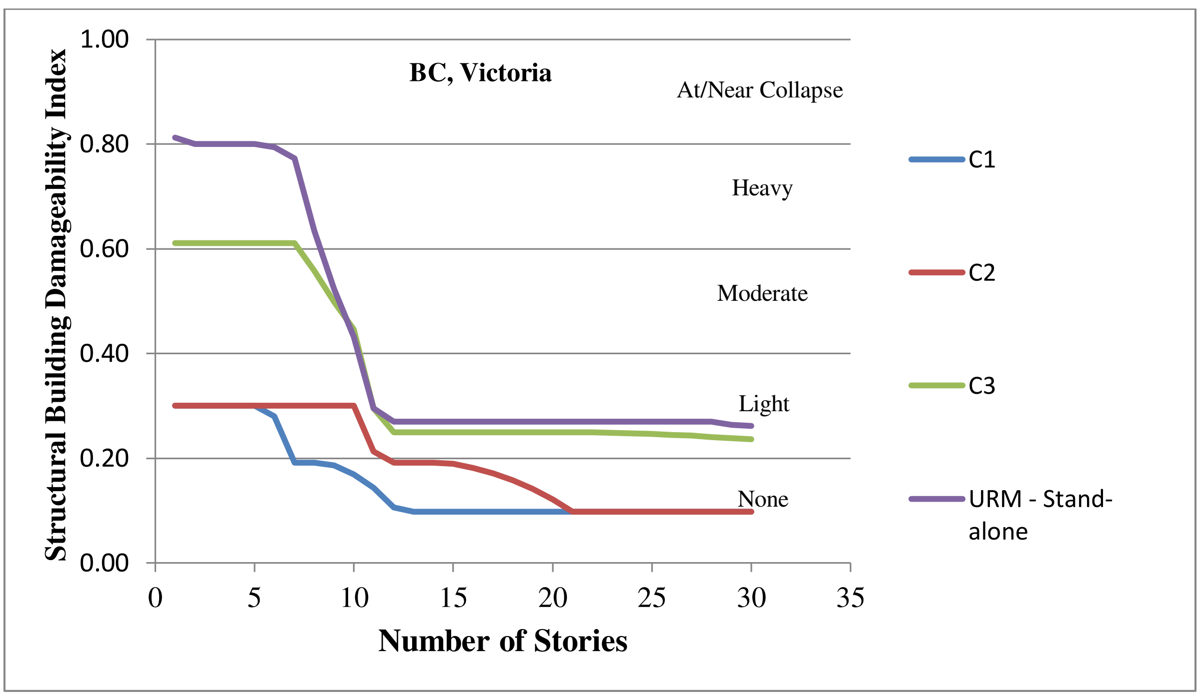


Figure 4-25: Comparison of Building Type for Structural Building Damageability of Structures Resting on Reference Site Soil Class C in Victoria, BC

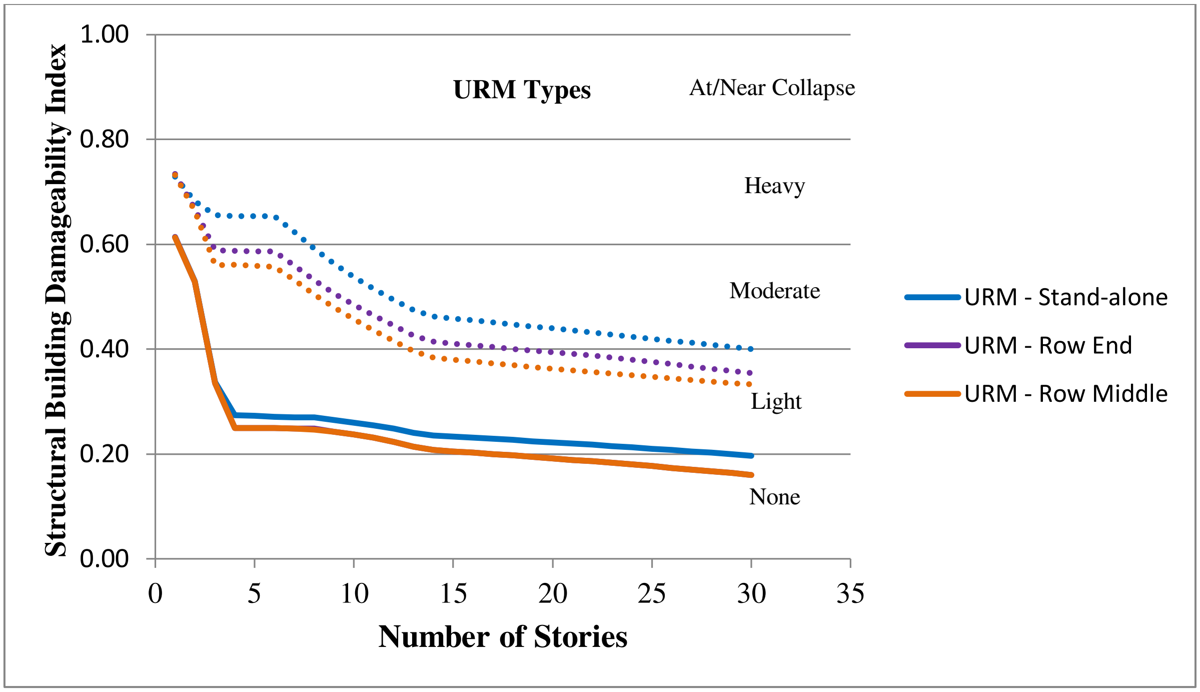


Figure 4-26: Comparison of URM Building Type for Structural Building Damageability of Structures Resting on Reference Site Soil Class C in Ottawa, ON

4.3 Model Verification

4.3.1 2011 Christchurch Earthquake

On February 22nd, 2011, New Zealand was struck by an earthquake of moment magnitude Mw = 6.2, resulting in 185 deaths (Elwood, 2013). The earthquake caused extensive damage in a large number of URM buildings located in the Central Business District (CBD) as summarized in Table 2-4 of Section 2.5.6. The aftermath of the earthquake resulted in the demolition of many URM buildings which were tagged as unsafe to enter (Ingham and Griffith, 2011a). Although the building stock in Christchurch had been subjected to previous large magnitude earthquakes, the Christchurch Earthquake was one the most catastrophic on record, with the URM building stock suffering wide-scale damage (Moon et al., 2012). The epicenter of the earthquake was located less than 10 km south-west of the CBD as illustrated in Figure 4-27.

Figure 4-27: Epicentre Distance to the Central Business District (CBD) of the 2011 Christchurch Earthquake in New Zealand

The reported peak ground acceleration (PGA) was measured up to be 2.2g in a valley approximately 1 km from the epicenter and 0.8g in the CBD. The horizontal ground motion records for the East-West and North-South directions of four stations in the Christchurch CBD are shown in Figure 4-28.

In terms of the geotechnical conditions of the built environment, softer soil profiles, defined as site class E in NBCC-2010 (site class D equivalent in New Zealand), were predominantly observed in the CBD of Christchurch (Elwood, 2013).

The mean earthquake spectrum of the four stations is illustrated in Figure 4-29 and compared to the UHS and design spectrum of Vancouver and New Zealand for two different return periods. It is clearly illustrated in Figure 4-29 that the mean earthquake spectrum sits fairly close to, and for the most part exceeds, the spectral acceleration of the UHS and the design spectrum. Although, the mean spectrum does not represent the spatial variability of seismic hazard, the spectral acceleration corresponding to a specific period can be used to quantify the site seismic hazard (Tesfamariam, 2008) and will be used in the analysis that follows.

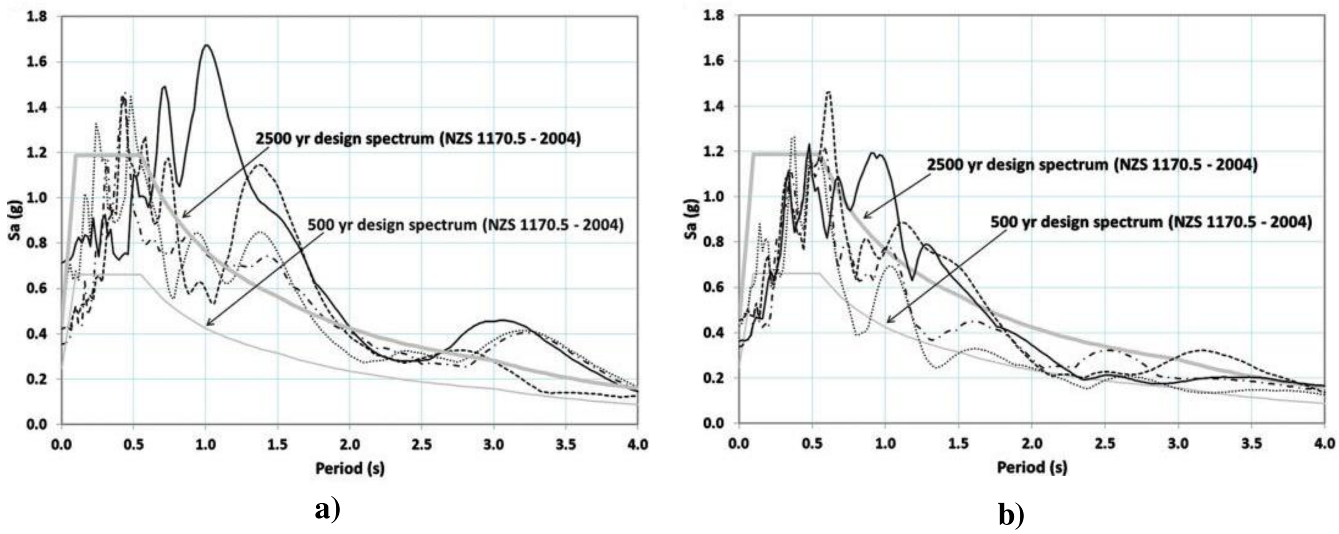


Figure 4-28: 5% damped spectra for ground motions recorded of the February 2011 Christchurch Earthquake in New Zealand for: a) East-West Direction b) North-south Direction

(Elwood, 2013)

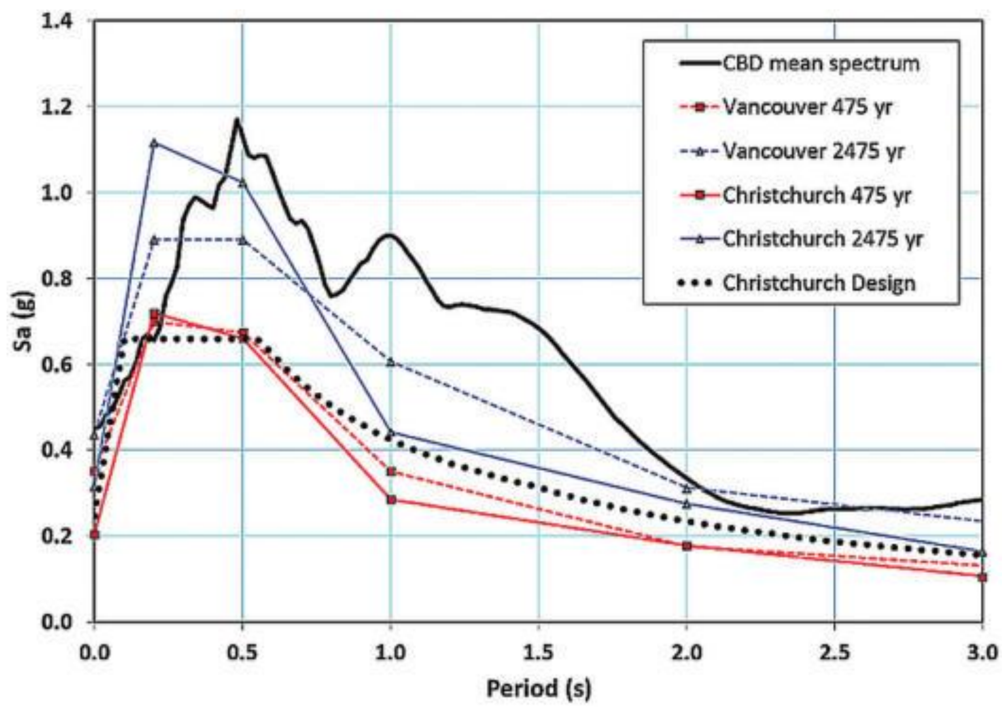


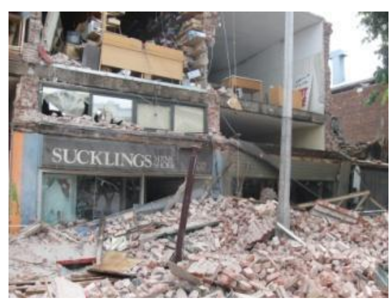
Figure 4-29: Comparison of mean spectrum for all horizontal ground motions of the February 2011 Christchurch Earthquake to the UHS design spectrum for Vancouver and Christchurch (soil class D assumed).

(Elwood, 2013)

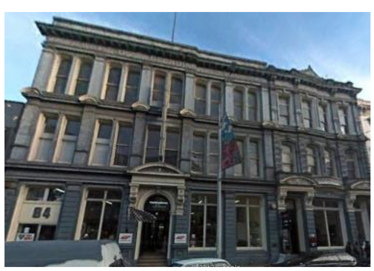
4.3.2 Validation of CanRisk using Selected Christchurch URM Buildings

Information relating to the condition of over 300 URM buildings in Christchurch has been provided by researchers from New Zealand (J. Ingham, K.Q. Walsh, L. Moon, personal communication, June 12, 2013). For the purpose of the CanRisk validation, fifteen buildings located in the CBD of Christchurch (seen in Figure 4-30 and Table 4-1) were selected and evaluated to assess structural damageability. The results are compared with the ATC-13 damage states reported by Ingham et al. (2011) following the earthquake. Buildings with varying damage states were selected ("light" to "destroyed") in order to cover a wide range of observed building damage.

Table 4-1: Pre and Post Earthquake photos of Selected Christchurch URM Buildings and their respective ATC-13 Damage Classifications

#	Pre-earthquake photo	Post-earthquake photo	ATC-13 Classification
1			Heavy
2			Major
3			Major

#	Pre-earthquake photo	Post-earthquake photo	ATC-13 Classification
4			Light
5			Moderate
6			Moderate
7			Major

#	Pre-earthquake photo	Post-earthquake photo	ATC-13 Classification
8			Heavy
9			Destroyed
10		N/A	Light
11			Destroyed

#	Pre-earthquake photo	Post-earthquake photo	ATC-13 Classification
12			Moderate
13			Major
14		N/A	Light
15			Heavy

The results of the analysis are summarized in Table 4-2. It is noted that the available data does not include sufficient information about the damageability of non-structural components; therefore results of a non-structural risk assessment are not presented. Furthermore, insufficient information is available to compute building importance/exposure; therefore a complete structural risk evaluation was not conducted and only a comparison of Structural Building Damageability is presented (see level 2 of Figure 3-6).

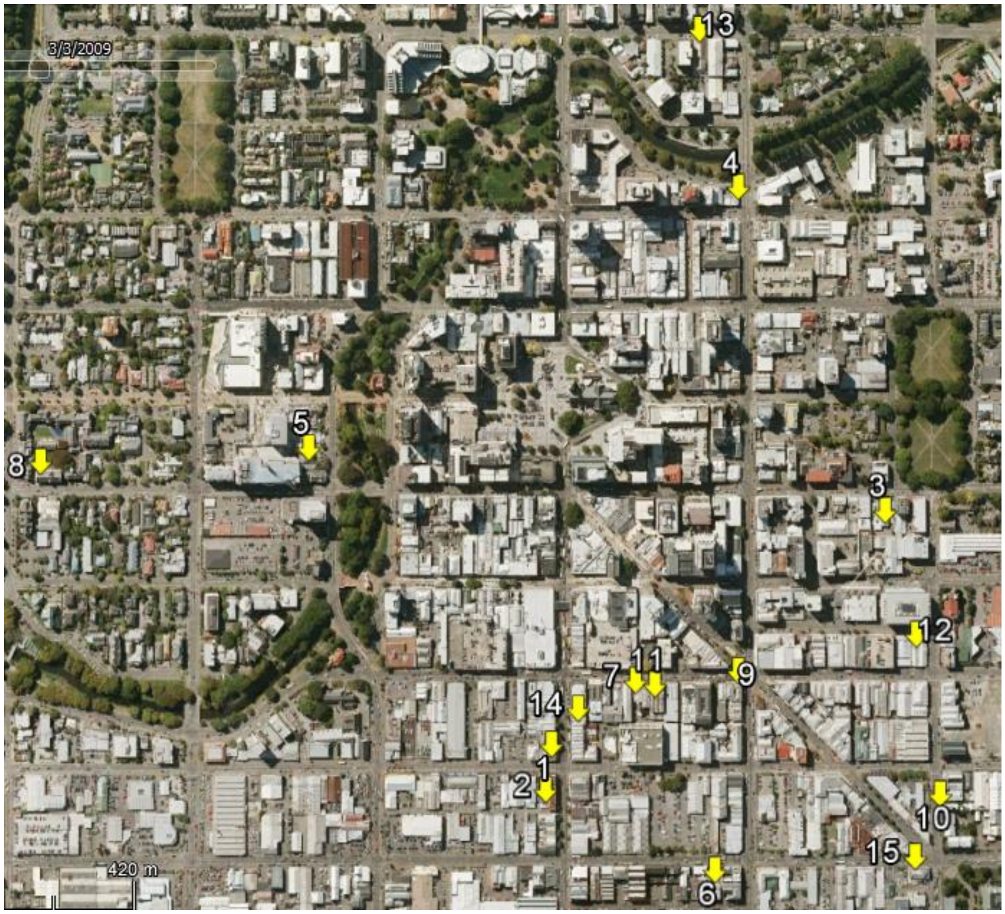


Figure 4-30: Evaluated URM buildings in the CBD

The buildings selected contain information relative to the year of construction, configuration, number of stories and whether or not the building had been retrofitted. The buildings that had been retrofitted are identified in Table 4-2 and contain either a type "A" retrofit, consisting of restraining non-structural elements to the structural components, or a type "B" retrofit, where new structural lateral force-resisting elements were added to the structure (Ingham and Griffith, 2011b). A building with an unknown year of construction is assumed to be built prior to 1940 as the CBD is known to be a heritage suburb of Christchurch. The construction quality, vertical irregularity and plan irregularity were identified through observations of photos of the building prior to the earthquake.

The estimated structural building damageability indices (I_S^{BD}), as computed using CanRisk, are summarized in Table 4-2, and compared with the ATC-13 damage states reported for the buildings. The results show relatively good agreement between the estimated structural building damageability values and those observed after the earthquake. In general the discrepancies fall within one "damage state" when compared with those reported in the ATC evaluations. It is noted that CanRisk uses a 5 level gradation scale to define building damageability (ranging from "none" to "at or near collapse"), whereas ATC-13 uses 7 damage state levels (ranging from "none" to "destroyed"), which can also explain some of the discrepancies.

Table 4-2: Evaluation Summary of URM structures in the CBD of the 2011 Christchurch Earthquake

Building Number	Building Address	Configuration	Number of Stories	Vertical Irregularity	Plan Irregularity	Spectral Acceleration	Distance from epicenter (km)	Construction Quality	Year of Construction	Structural Damageability Index (I^{BD})	Structural Damageability (CanRisk)	ATC-13 Damage State
1	623-629 Colombo St	Row-Middle	2	M	VL	0.65	7.48	AVG	1890 (retrofit A)	0.61	Heavy	Heavy
2	603-615 Colombo St	Row-Middle	2	H	L	0.65	7.45	AVG	1906	0.72	Heavy	Major
3	208 Hereford St	Stand-alone	2	VL	VL	0.65	7.41	AVG	1865	0.65	Heavy	Major
4	287 Manchester St	Row-End	1	L	VL	0.52	7.87	GD	unknown	0.4	Light	Light
5	107A Cambridge Terrace	Row-End	1	VL	M	0.52	7.98	PR	1875 (retrofit A)	0.47	Moderate	Moderate
6	222 St Asaph St	Row-Middle	1	L	VL	0.52	7.22	AVG	unknown (retrofit A)	0.43	Moderate	Moderate
7	80 Lichfield St	Row-End	3	L	VL	0.77	7.44	AVG	1881	0.84	At/Near Collapse	Major
8	1-3 Hereford St	Row-Middle	3	VL	VL	0.77	8.22	GD	1910	0.78	Heavy	Heavy
9	116 Lichfield St	Row-End	3	H	M	0.77	7.39	PR	unknown	0.92	At/Near Collapse	Destroyed
10	116 Madras St	Stand-alone	2	VL	VL	0.52	7.09	GD	1876 (retrofit A&B)	0.33	Light	Light
11	84 Lichfield St	Row-End	3	VL	VL	0.77	7.45	AVG	1890 (retrofit A)	0.84	At/Near Collapse	Destroyed
12	141 Lichfield St	Row-End	1	VL	M	0.52	7.26	PR	1903 (retrofit A)	0.53	Moderate	Moderate
13	146 Kilmore St	Stand-alone	2	L	L	0.65	8.05	PR	1929	0.72	Heavy	Major
14	644 Colombo St	Row-Middle	2	L	M	0.65	7.5	AVG	unknown (retrofit A&B)	0.6	Moderate	Light
15	270 St Asaph St	Stand-alone	2	L	L	0.65	7.06	AVG	unknown (retrofit A&B)	0.61	Heavy	Heavy

Chapter 5
Urban RAT Data Collection of Buildings in the City of Ottawa

5.1 General

A general building inventory and its spatial distribution and variability are key parameters needed for earthquake loss assessment and risk management. Although data collection of building information may be a daunting task, new tools have been developed to provide the information and building data necessary for earthquake loss estimations. The Urban Rapid Assessment Tool (Urban RAT) is designed for the rapid collection of building data in urban centres. The Geographic Information System (GIS) based assessment desktop and mobile toolset allows for intense data collection and revolutionizes the traditional sidewalk survey approach forcollecting building data (Sawada et al., 2013). The following chapter describes the application of Urban RAT software to downtown core of the City of Ottawa. Information pertaining to the condition of existing buildings, including the spatial distribution and percentage breakdown of construction type, local soil conditions, occupancy class, year of construction, and irregular building configurations relevant to seismic risk assessments are presented. Results are presented for the general building stock and unreinforced masonry building inventory.

5.2 Urban Rapid Assessment Tool (URAT)

The Urban Rapid Assessment Tool (URAT) suite modernizes the way building surveys are conducted. Rather than the traditional pen and paper sidewalk survey, the Urban RAT tool exploits the use of computers, web services and portable electronics in order to obtain and collect site specific building information.

Urban RAT is an ArcGIS-Google-Android system that contains two components: an in-lab application (add-in) built for ArcGIS 10.x within the .Net framework (in order to integrate ArcGIS and the Google API) and second, an on-site (Google Android) app that collects positional and visual information in addition to inputs that contain the same data. The on-site application data can be synchronized with the main ArcGIS database.

Within the lab, using a MS Windows PC with ArcGIS 10.x installed, the user is presented with a new toolbar called URAT (Figure 5-1)

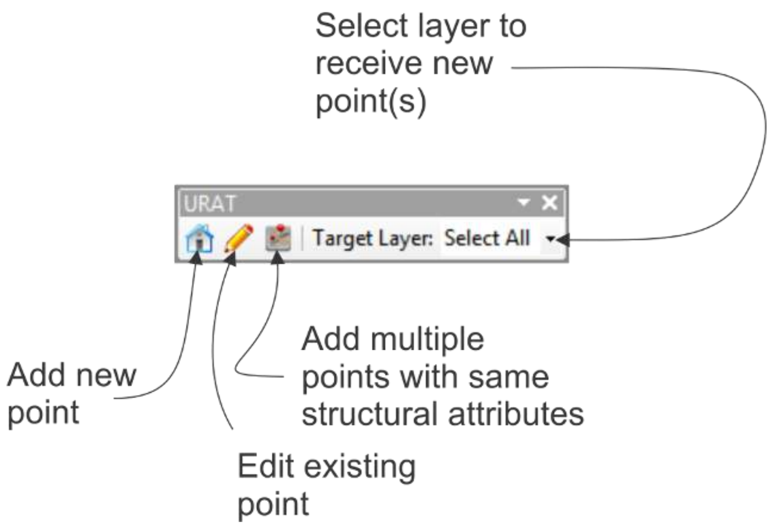
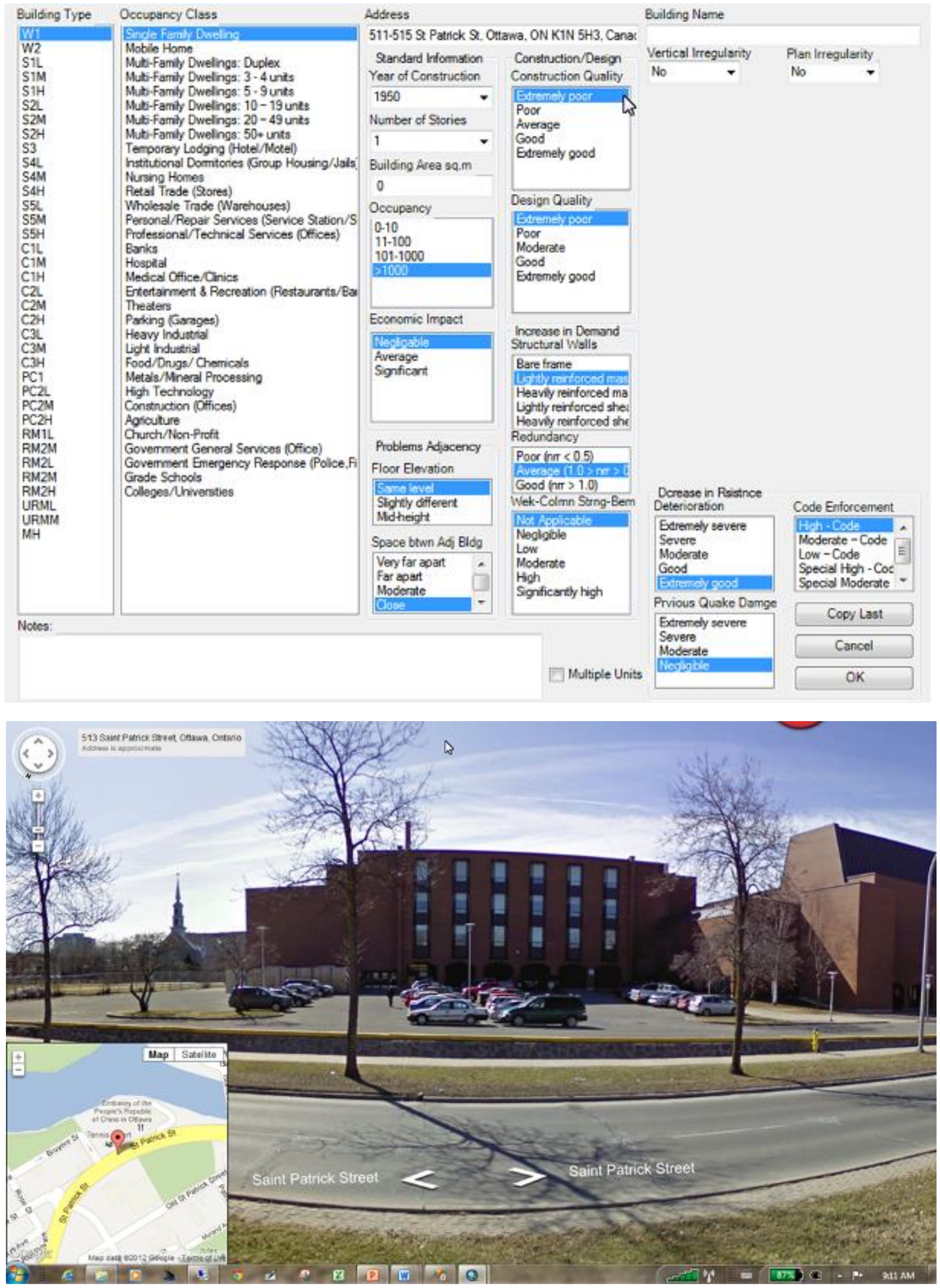


Figure 5-1 Urban RAT toolbar in ArcGIS 10.x

(from Sawada et al., 2013)

Using this toolset, the user selects by simply clicking on a building represented on a satellite image within ArcGIS. This initiates two windows. The first shows the form with building parameters to be entered (Figure 5-2a) and the second window opens Google StreetView within ArcGIS at the location of the building that was selected (Figure 5-2b). This allows the assessor to examine the structure from many angles and enters parameters on the form. Once the form is complete the data is automatically saved into a new data layer with a point at the location of the assessment.

a)



b)

Figure 5-2 a) Building assessment form in Urban RAT; b) Google StreetView within Urban RAT and within ArcGIS open at location of building to be assessed

(from Sawada et al., 2013)

Urban RAT suite's framework is designed to incorporate roughly 30 structural parameters. Table 5-1 presents Urban RAT's parameters for assessment that are based on FEMA 154 (2002) and FEMA 310 (1998). The first theme ([1] General information) provides the basic information related to a buildings characteristics and structural system. The second and third themes ([2] Increase in Demand and [3] Decrease in Demand) represent endogenic engineering parameters which influence building vulnerability during earthquake events. The final theme ([4] Issues of Adjacency) incorporates an imperative exogenic factor which can affect structural performance during earthquake ground shaking. Themes [1]-[4] are required for high resolution earthquake loss estimation studies. These variables and their respective values are presented to the user on the main URAT interface (Figure 5-2a).

Table 5-1: URAT Theme Parameters for Assessment

[1] General	
Building Type	Year of Construction
Address	Number of Stories
Name of Building	Occupancy Class
Vertical Irregularity	Occupancy
Plan Irregularity	Economic Impact
Construction Quality	Design Quality
[2] Increase in Demand	
Structural Walls	Weak Column-Strong Beam
Redundancy	
Plan Irregularity	
Diaphragm Continuity	Torsional Irregularity
Re-Entrant Corners	
Vertical Irregularity	
Short Column Effect	Soft Story
(Captivated Column)	Weak Story
[3] Decrease in Resistance	
Deterioration (e.g. Corrosion)	Code Enforcement
Damaged from Previous Earthquake	
[4] Issues with Adjacency	
Floor Elevation	Space Between Adjacent Buildings

In some cases, the assessor will find that the Google StreetView is insufficient for assessment. As such, a mobile version of the virtual site assessment software can be used and will run on any certified Google Android tablet. There is no need to have an active wireless internet connection (Wi-Fi, 3G, 4G or otherwise) with Urban RAT mobile in order to make full use of the tablet's GPS and mapping functions. In Urban RAT mobile (Figure 5-3), all data is stored locally on the device as XML and CSV files which can be easily uploaded to the main ArcGIS program when the user returns to the desktop.

For further information on details of the development and use of the Urban RAT suite refer to Sawada et al. (2013).

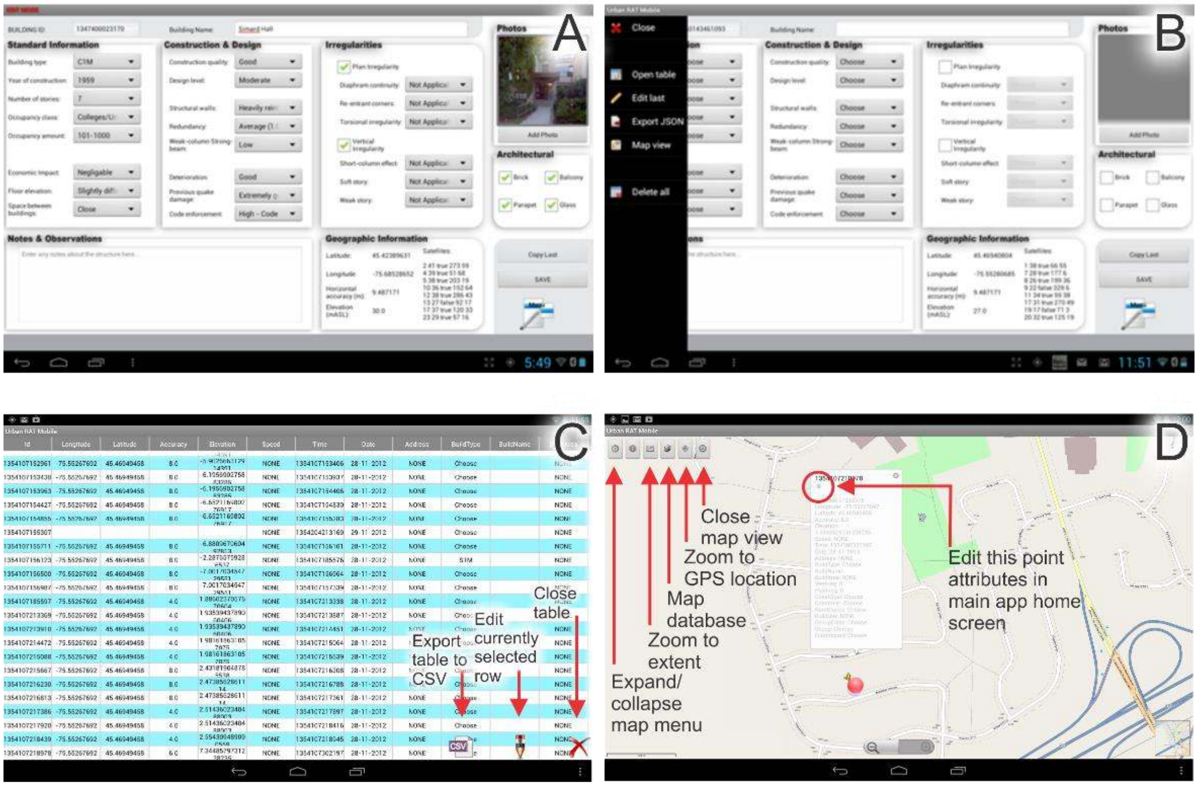


Figure 5-3: Urban RAT mobile: a) Main assessment screen, variables as in Table 1; b) Main menu used to switch between data entry screen, map and data table; c) Data table of stored assessment locations. User can edit or export to comma separated values file (CSV); d) Map of assessment area. User can plot all assessed points, select individual points for editing and see current location on map using GPS receiver in tablet.

5.2.1 Urban RAT Building Inventory

The application of the Urban RAT to the downtown core of the City of Ottawa began in the summer of 2011 in order to create a database and collect building information to eventually allow for the seismic risk assessment of buildings in the City of Ottawa. The following eight neighbourhoods were the primary focus for assessment: Centretown, West Centretown, Byward Market, Lowertown, Sandy Hill, Ottawa East, Glebe – Dows Lake and Ottawa South. Currently, the number of buildings assessed consists of 13,038 buildings seen in Figure 5-4. In general, most downtown neighbourhoods in the City of Ottawa contain a combination of historical and modern buildings. Within the building inventory a total of

1,465 (~11.2% of total inventory) buildings were classified as URM construction. Table 5-2 presents the approximate percent coverage (number of buildings assessed / total building inventory) for each neighbourhood. On average, a total coverage of approximately 78% of the eight major neighbourhoods has been completed to date. Although data collection is ongoing, the number of buildings assessed has been limited in part due to the necessity for in-field assessments in the case of complex structural systems, the lack of availability of supplementary material that assist during assessments (e.g. lack of fire insurance plans for some buildings), limitations of Google StreetView and new construction.

The sections that follow provide information relevant to the building inventory obtained using Urban RAT. Information pertaining to the spatial distribution and percentage breakdown of construction type, local soil conditions, occupancy class, year of construction, and irregular building configurations relevant to seismic risk assessments is presented. Results are presented for the general building stock and unreinforced masonry building inventory.

Table 5-2: Percent Coverage of Major Neighbourhoods

Neighbourhood	Coverage %
Centretown	83
Byward Market	78
Glebe-Dows Lake	62
Lowertown	75
Ottawa East	100
Ottawa South	79
Sandy Hill – Ottawa East	99
West Centretown	46

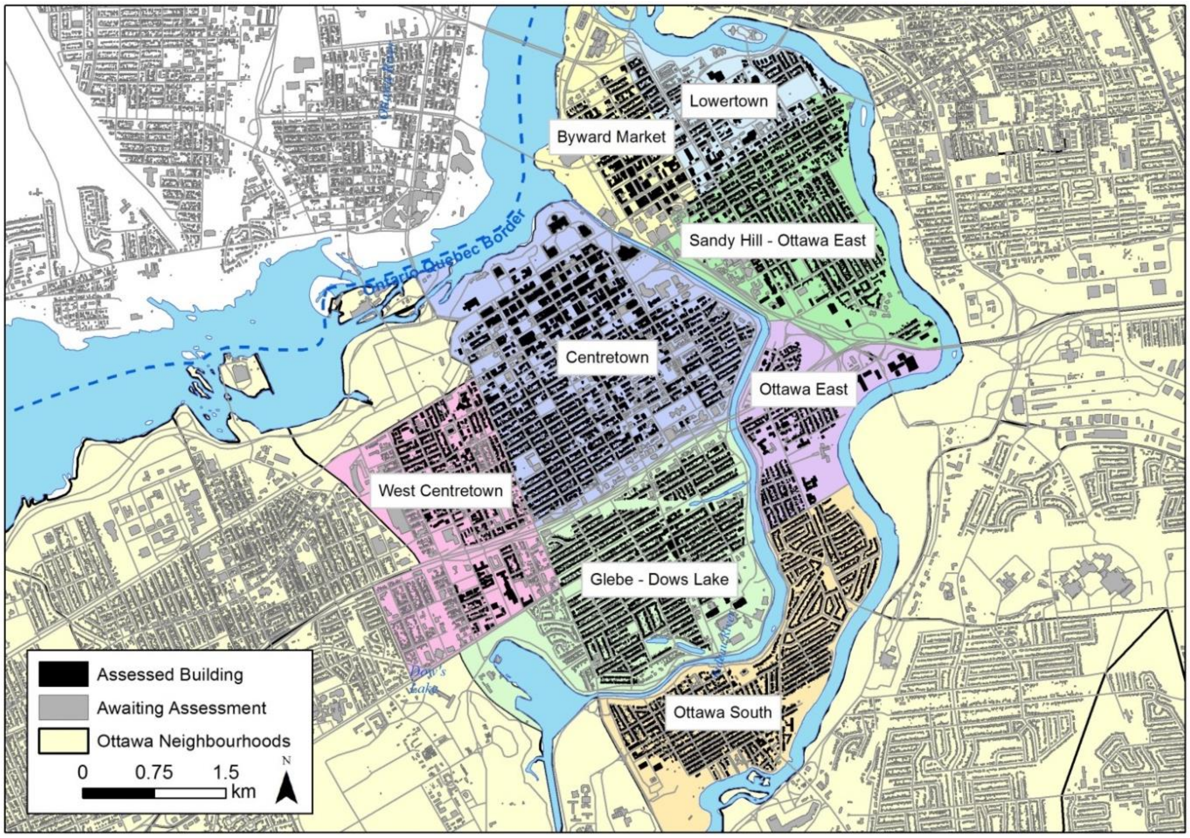


Figure 5-4: Study Area

5.2.2 Construction Type

The construction type of a building influences its seismic performance. Figure 5-5 presents the spatial distribution of construction type within the study area. A building is expected to exhibit a brittle or ductile response in the incident of an earthquake as a function of the engineering and type of material utilized for construction. Modern engineered buildings in seismic areas are specifically designed to withstand expected lateral loads and perform better than non-engineered buildings. At the same time, the seismic performance of engineered buildings will depend on earthquake hazard and the level of building code. Typically reinforced concrete or steel buildings fall in the category of engineered buildings. More recent timber and reinforced masonry construction can also be included in this category due to the development of design standards; however older unreinforced masonry buildings can be considered non-engineered construction. For the buildings assessed in this study, reinforced concrete, reinforced masonry and steel buildings were considered engineered construction while unreinforced masonry and wood buildings were classified as non-engineered. As shown in Figure 5-6, the majority of buildings assessed in this study can be classified as non-engineered buildings while approximately 6.1% of the building stock can be categorized as engineered. Of particular concern is the large inventory of unreinforced masonry buildings (11.2%), a structural material which has consistently demonstrated poor performance in past earthquakes. In addition, it is noted that a large inventory of residential single family dwellings built of wood were included in the assessment (82.7%).

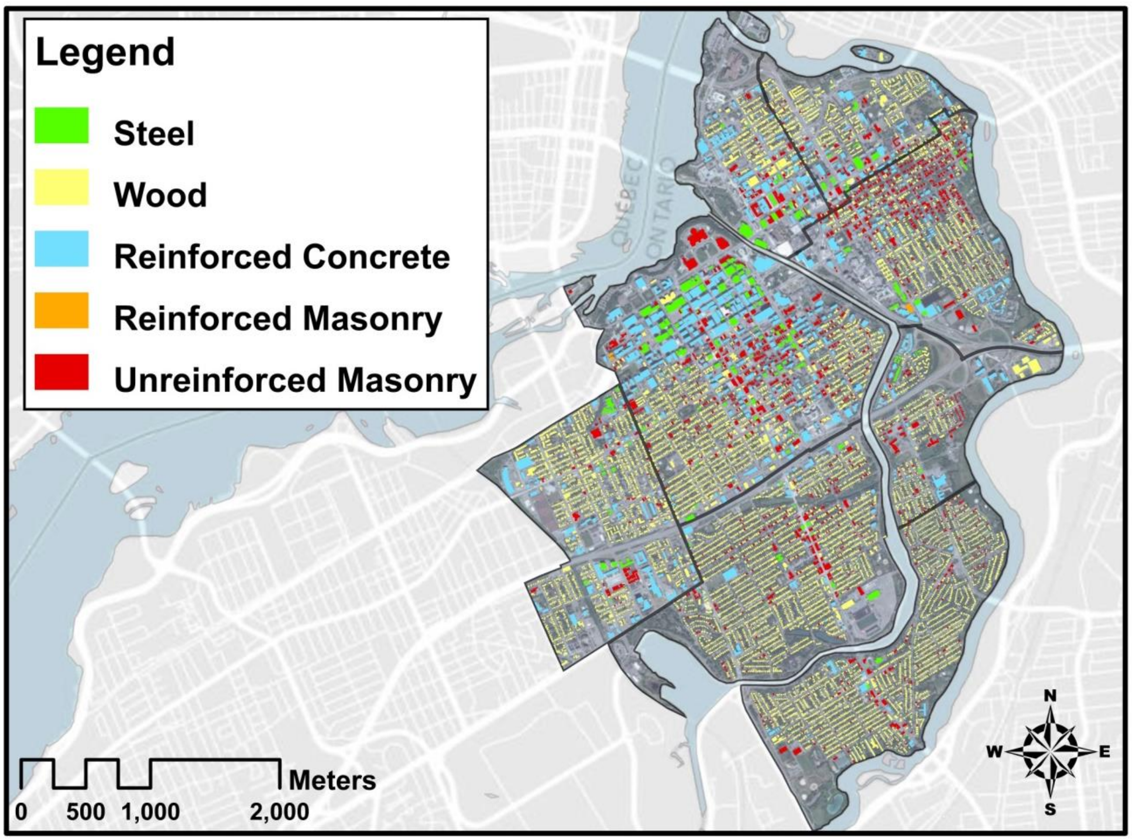


Figure 5-5: Spatial Distribution of Construction Type of Buildings in Ottawa, Ontario

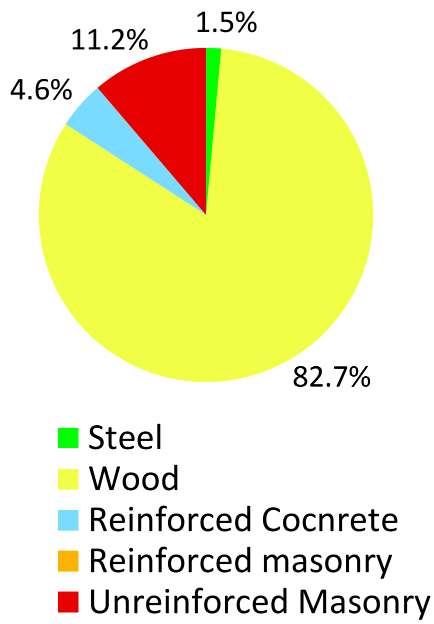


Figure 5-6: Percent Breakdown of Construction Type of Buildings in Ottawa, Ontario

5.2.3 Site Soil Classification

In addition to the structural make-up of any given building, another important variable in assessing seismic vulnerability is the soil condition (surficial geology) in which it rests upon. The local ground conditions in which a building rests is a key indicator to ground shaking intensities. Figure 5-7 illustrates the spatial variability of site soil classification for each building (Hunter et al., 2010; Motazedian et al., 2011). Approximately 35% (Figure 5-8a) of the buildings assessed are constructed on soft or stiff soil conditions (class D & E) while the remainder of the building stock are constructed on hard rock or very dense soil profiles (class A, B, C). With respect to the URM building stock, over 55% the URM buildings stock are located on site soil classification D & E (Figure 5-8b). These soil types include conditions of lower shear wave velocity that increase the strength of ground shaking of an earthquake (Williams et al., 1997).

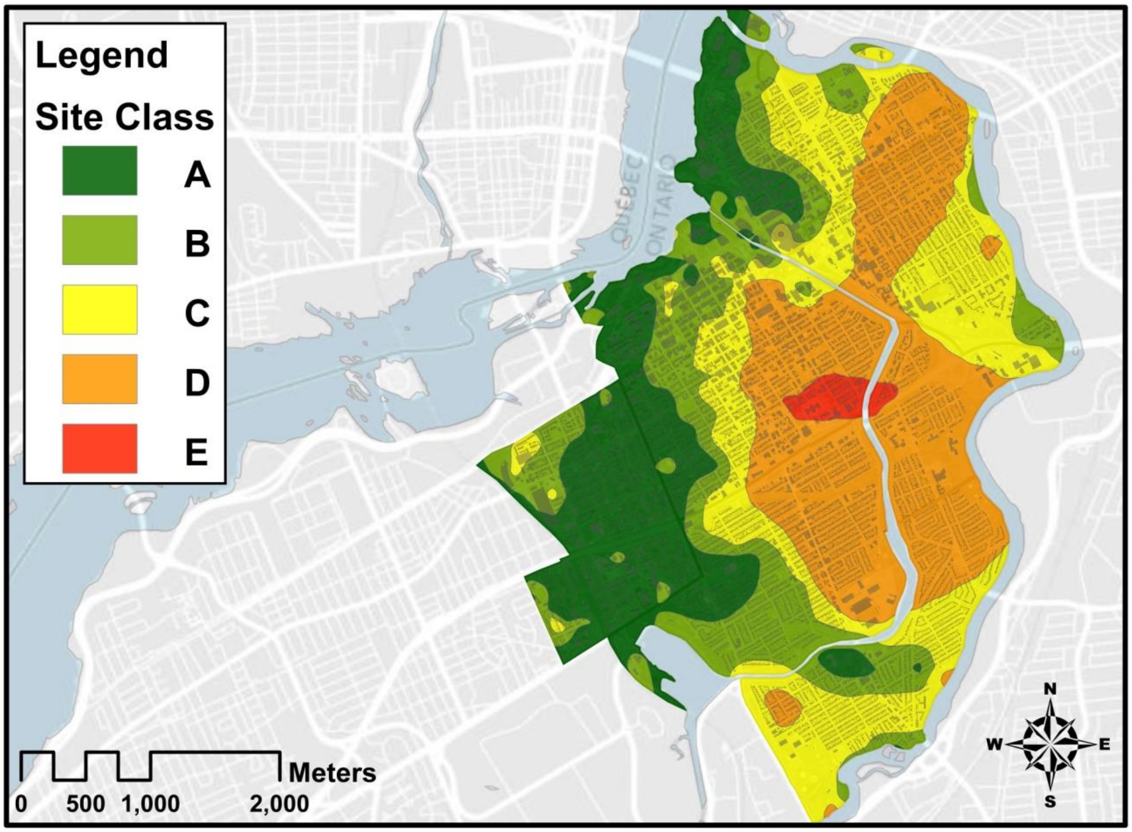
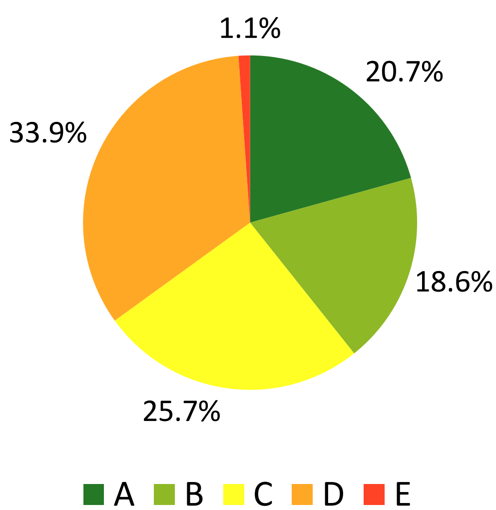
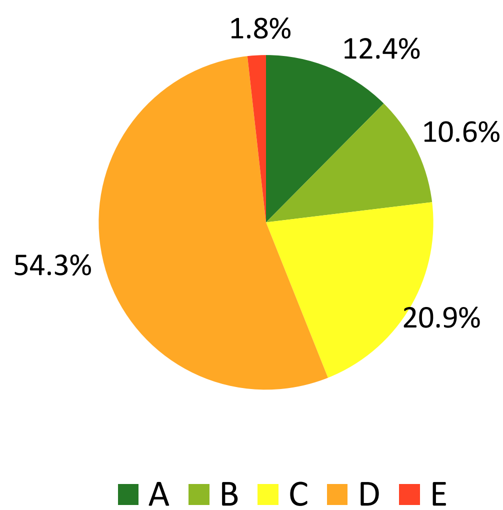


Figure 5-7: Spatial Distribution of Soil Classification of Buildings in Ottawa, Ontario



a) Total Inventory



b) URM Inventory

Figure 5-8: Percent Breakdown of Soil Classification: a) Total Inventory, b) URM

5.2.4 Year of Construction

As mentioned previously, one very important factor affecting seismic performance is the building year of construction. The year of construction of a building delineates older construction from the new, more modern practices. Figure 5-9 illustrates that the downtown core of the City of Ottawa is in fact an older, historical part of the city with a majority of buildings built prior to 1940 (Figure 5-10a). The original design drawings of a building and/or supplementary information such as census dissemination area age of construction or tax records can be useful in determining the year of construction. The year of construction, when considered with historical development of building code and seismic design criteria, can provide insight on the seismic design loads and level of seismic design and detailing of a building. In order to determine seismic vulnerability, it is important to understand the development of the seismic design code provisions over the years. According to NIBS (1999) and Tesfamariam and Saatcioglu (2008), the level of building code can be divided into three distinct states for North America: low code (pre 1941), moderate code (between 1941 and 1975) and high code (post 1975). Considering this breakdown, analysis of the building inventory reveals that the vast majority of the building stock was built prior to 1940, indicating that most of the buildings are in compliance with low code provisions. The year of construction of unreinforced masonry buildings is an indicator of the probable performance in the event of an earthquake. Figure 5-10b illustrates that the majority (over 80%) of URM buildings in the downtown core were built prior to the 1940s where no stringent seismic requirements were in place for the construction of URM. This indicates the importance of evaluating the URM building stock in order to mitigate against seismic risk associated with this construction type by means retrofit or rehabilitation to assure a proper level of seismic safety in accordance with modern code requirements.

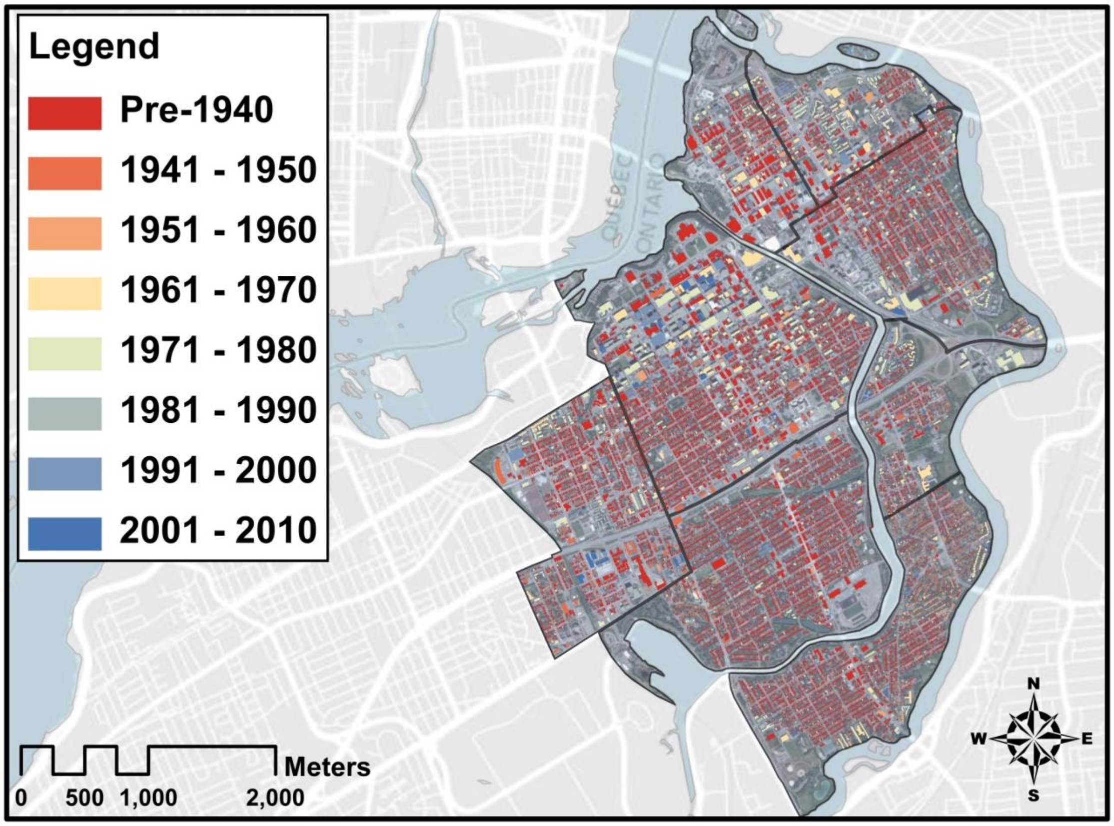
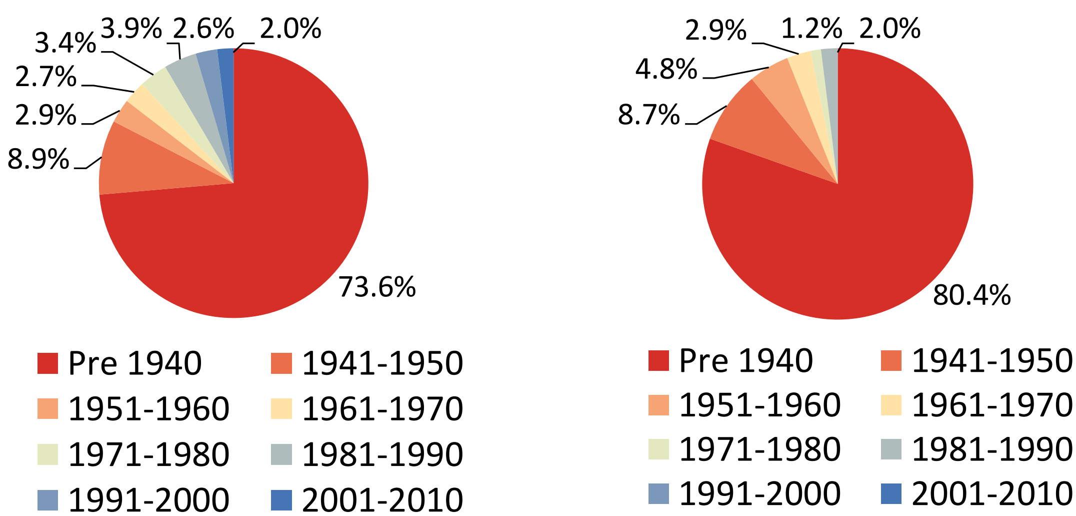


Figure 5-9: Spatial Distribution of Year of Construction of Buildings in Ottawa, Ontario



a) Total Inventory **b) URM Inventory**

Figure 5-10: Percent Breakdown of Year of Construction: a) Total Inventory, b) URM

5.2.5 Building Importance

The level of importance of a building can be established on the basis of the building's occupancy and use. Observations from previous earthquakes have emphasized that certain critical facilities such as hospitals should be designed to remain operational during and after an earthquake. Figure 5-11 illustrates the spatial variance of occupancy class within the building inventory. The NBCC classifies an importance factor dependent on the building's occupancy when determining the total seismic base shear the building is designed to resist. The categories include normal, high and post-disaster importance classifications. High importance structures include schools and community centres that are able to house a large number of individuals including young children. Post-disaster buildings include hospitals and emergency response facilities that are required to remain operational in the event of a disaster. Normal importance buildings include all other buildings that do not fall in the high or post-disaster categories (NRCC 2010). Within the building stock, high importance and post-disaster buildings represent 0.7% and 0.1% of the total building stock respectively (Figure 5-12a). As displayed in Figure 5-12b, a number of educational facilities, emergency response and medical facilities built of URM are located in the downtown core. As URM is the most vulnerable form of construction, it is important to evaluate the performance of these buildings. Overall, the inventory reveals there is a rich mixture of occupancy class within the building stock with a majority of URM consisting of single or multifamily dwellings, temporary lodging and general service buildings (retail stores), however an important stock of URM buildings also fall in the post-disaster and high importance categories.

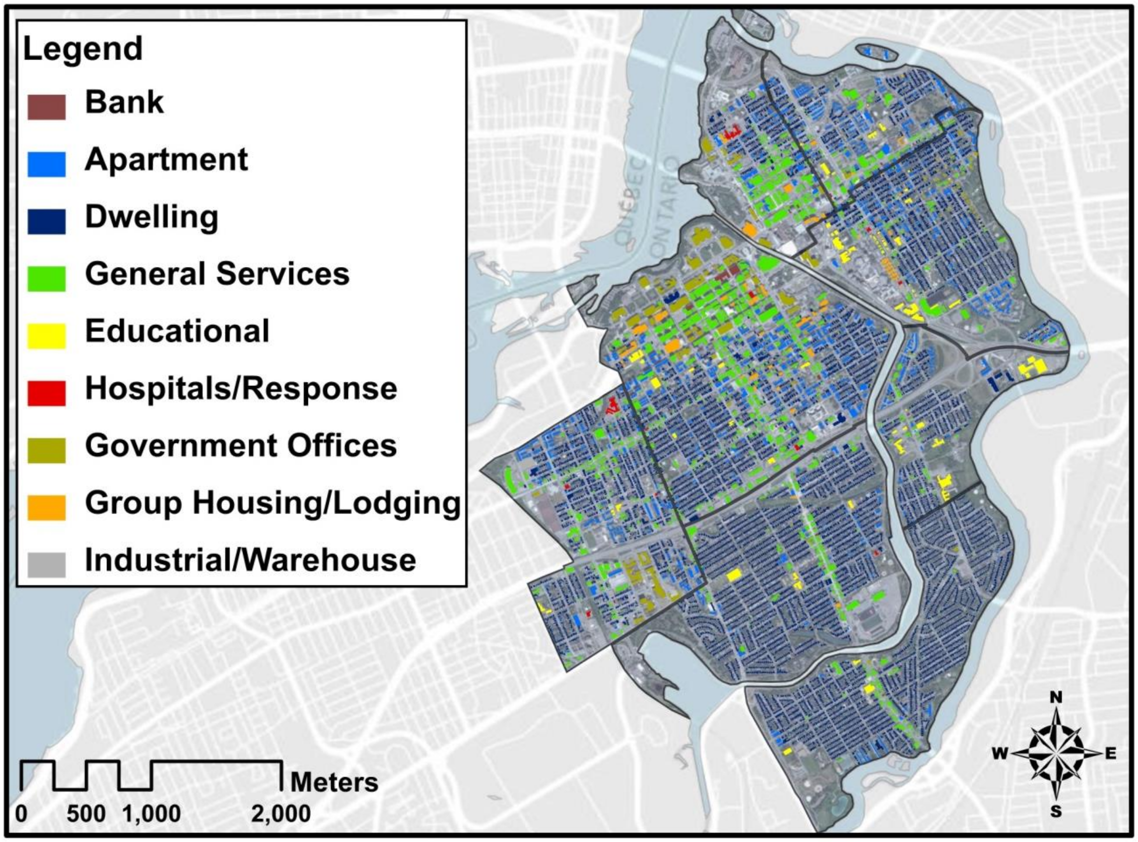
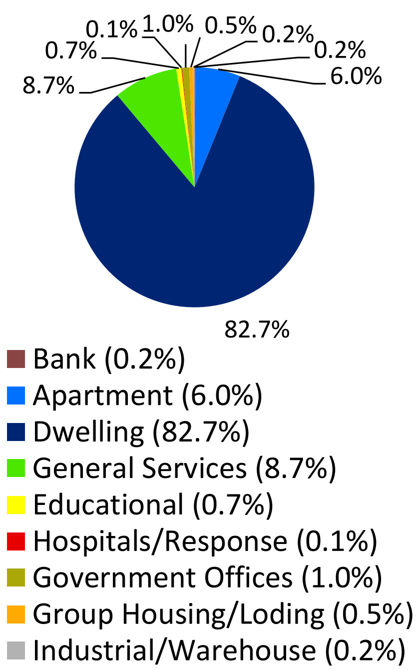
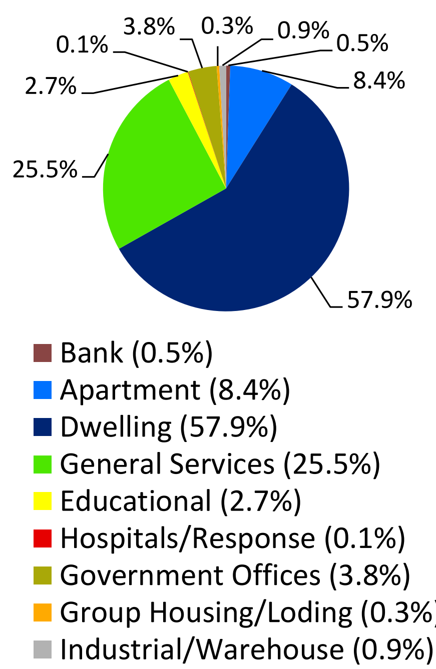


Figure 5-11: Spatial Distribution of Occupancy Class of buildings in Ottawa, Ontario



a) Total Inventory



b) URM Inventory

Figure 5-12: Percent Breakdown of Occupancy Class: a) Total Inventory, b) URM

5.2.6 Building Irregularity

Performance of buildings in past earthquakes has demonstrated that buildings with irregular configuration or irregular distribution of structural properties can cause an increase in seismic demand, leading to a greater degree of damage and greater risk of failure of a building (Tesfamariam & Saatcioglu 2008). Therefore, in the development of the NBCC seismic provisions, rules have provided for the classification of buildings into various building irregularity categories as a function of asymmetries (NRCC 2010). Accordingly, these parameters that evaluate structural irregularities have been accounted for in the building inventory and buildings within the inventory have been classified as regular and irregular in terms of configuration as seen in Figure 5-13. The two principal types of irregularity assessed include plan and/or vertical irregularities of a structure with 8.6% of the downtown building stock classified as containing irregular configurations (Figure 5-14a). The most common type of irregularity found in URM buildings are re-entrant corners caused by asymmetrical plan configurations and setbacks. In addition, a number of URM buildings contain first floor retail stores with one or two levels of family dwellings located above, particularly buildings along Bank Street – a street that is considered historical, a major shopping and business district in the City of Ottawa. This building configuration typically results in a soft story effect due to large display windows and storefront openings. Figure 5-14b illustrates that approximately 35% of the URM building stock contain an irregular structural configurations which include plan irregularity, vertical irregularity or a combination of the two aforementioned irregularity types. The presence of irregularities in this hazardous construction type results in increased seismic demand and can result in increased seismic vulnerability and the overall likelihood of failure.

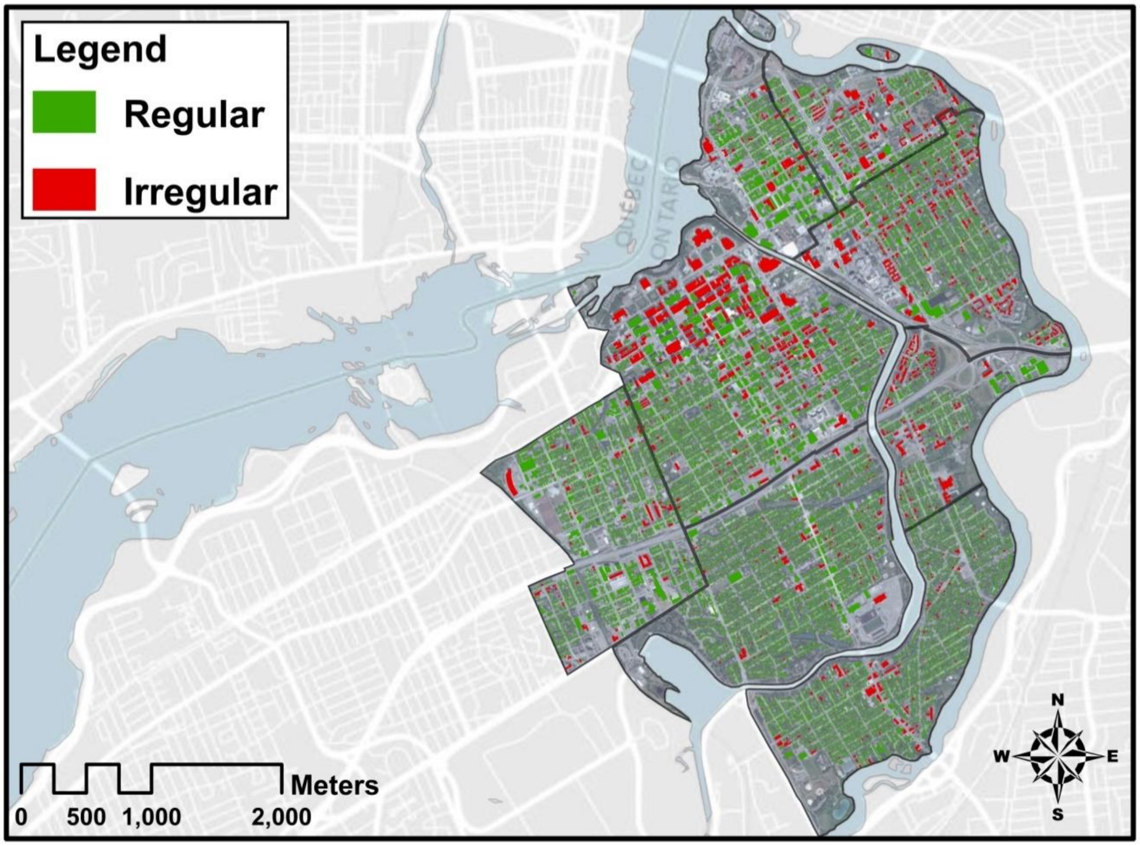
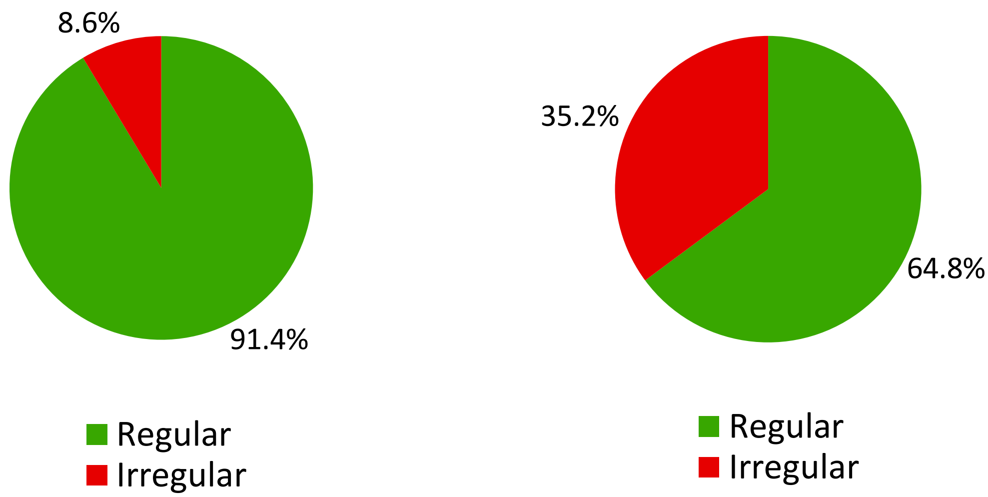


Figure 5-13: Spatial Distribution of Regular and Irregular buildings in Ottawa, Ontario



a) Total Inventory

b) URM Inventory

Figure 5-14: Percent Breakdown of Regular and Irregular Buildings: a) Total Inventory, b) URM

Figure 5-15 provides a detailed breakdown of the irregularities of the URM portion of the inventory evaluated using Urban RAT. The irregularities considered included: diaphragm discontinuity, re-entrant corners, torsional irregularity as well as soft story and weak story irregularities. Considering all irregularity types, the most common irregularity for URM buildings Ottawa was the re-entrant corner irregularity, followed by torsional irregularity and soft story irregularities.

In terms of re-entrant corners, the majority of URM buildings assessed were found to have "low" to "moderate" re-entrant corners. This type of irregularity is found if there is setback in the plan configuration that may cause stress concentration at the corner of two different spans of the building.

URM buildings having non-symmetrical configurations may cause torsional irregularity. URM buildings were assessed based on the characteristics of the plan layout and it was found that a number of buildings had this type of irregularity, with irregularity ranging from very low to high.

Many retail URM buildings (such as the ones found on Bank Street) have open fronts and this can cause evaluated a soft story vertical irregularity. The results of the analysis show that a number of URM buildings having soft stories, with irregularities ranging from very low to moderate.

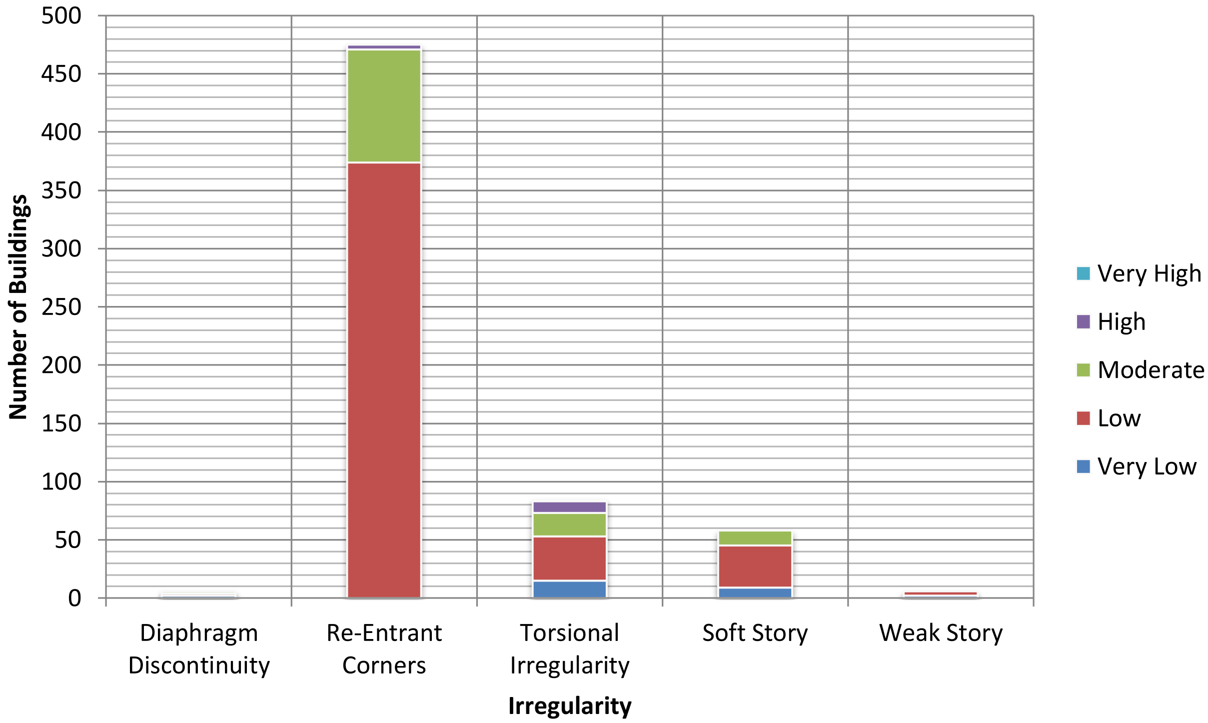


Figure 5-15: Detailed breakdown of URM irregularities

Chapter 6
CanRisk Seismic Risk Assessment of Buildings in the City of Ottawa

6.1 General

In the Ottawa-Gatineau region, continuous urban growth puts ever greater populations and infrastructure at risk to seismic disturbance (Lamontagne, 2010). According to seismologists, the Ottawa-Gatineau region has the 3rd largest seismic hazard in Canada based on historical earthquake records. With an ever growing population and concentrated building stock at risk, there is an urgent need to assess the seismic risk and vulnerability of buildings in this region. An understanding of seismic risk affecting the Ottawa-Gatineau region can provide knowledge to support effective actions by decision makers and increase preparedness in order to mitigate potential seismic related losses.

This chapter presents a summary of the seismic risk assessment of a large inventory of unreinforced masonry and reinforced concrete buildings in the City of Ottawa. Using data collected from the Urban RAT building inventory, the structural risk (SR) is assessed using CanRisk. Thereafter the detailed evaluation and overall building risk (OBR) of a number of URM buildings is presented.

6.2 CanRisk Regional Seismic Risk Assessment

The building inventory collected using the Urban RAT from section 5.2.1 is used to conduct a regional structural risk assessment for URM and RC structures in Ottawa, Ontario. The structural building damageability and structural risk is evaluated for 1,465 URM and 580 RC structures.

6.2.1 Site Seismic Hazard

The uniform hazard spectrum (UHS) can be formed using the seismic data for Ottawa, Ontario and identifying a building's site soil condition as defined in Table 3-3 and Table 3-4 respectively. The spectral acceleration of a building can then be established by using the NBCC-2010 empirical formulas for building period (see Table 3-5). Based on the site condition and structural period of the buildings assessed in section 5.2.1, the spectral acceleration for the City of Ottawa building inventory was computed as illustrated in Figure 6-1 and Figure 6-2a. When considering the building inventory in general, approximately 1.0% of building's were found to have a spectral acceleration of 0.10g or less, whereas over 50% of the building stock was found to have a spectral acceleration of 0.60g or greater. A large portion of the building inventory contains one-to-two story structures with low structural periods resulting in a high spectral acceleration value depending on the site soil condition that the building rests upon. When considering the URM portion of the building inventory, over 65% of the URM building are sitting on softer soil profiles (Class D and E) and have a spectral acceleration value of 0.60g or higher as seen in Figure 6-2b.

The spectral acceleration value for the building inventory provides a good indication of site seismic hazard when conducting seismic risk assessments as discussed in section 3.5 and will be used in order to determine the structural building damageability (SBD) and structural risk (SR) for the URM and RC portion of the building inventory (see sections 6.2.2. and 6.2.3).

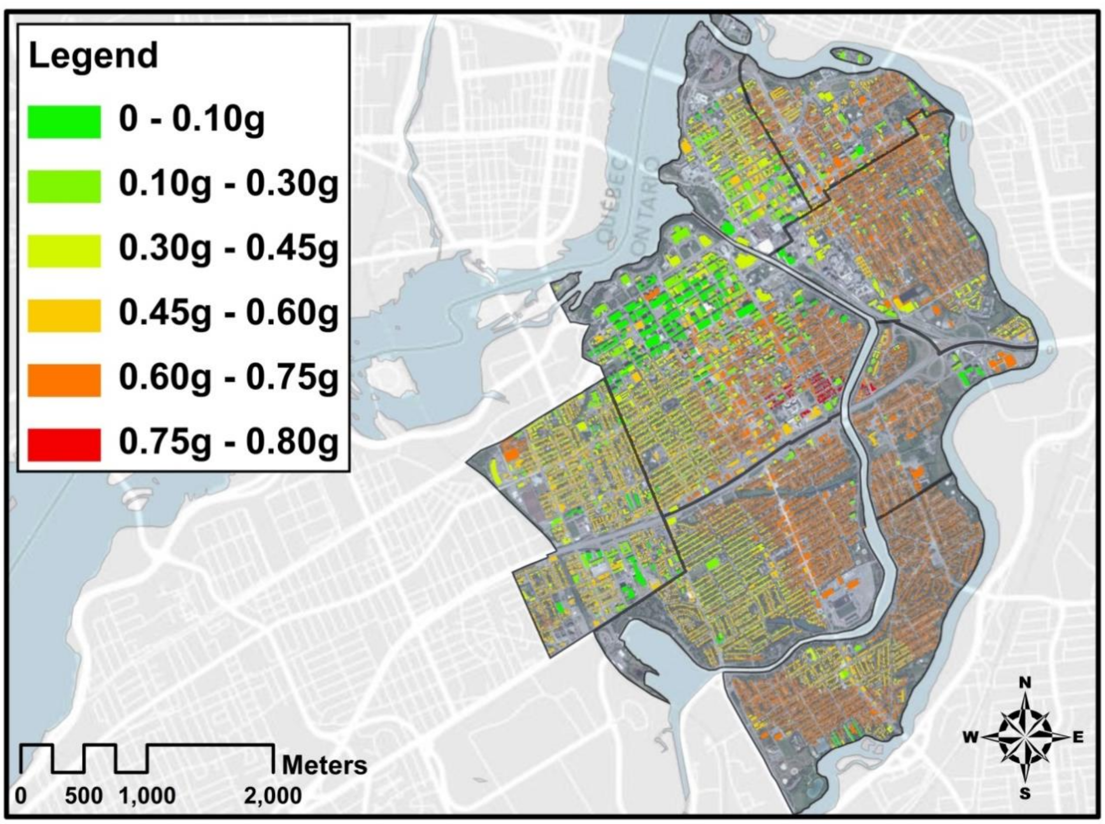


Figure 6-1: Spatial Distribution of Spectral Acceleration of Buildings in Ottawa, Ontario

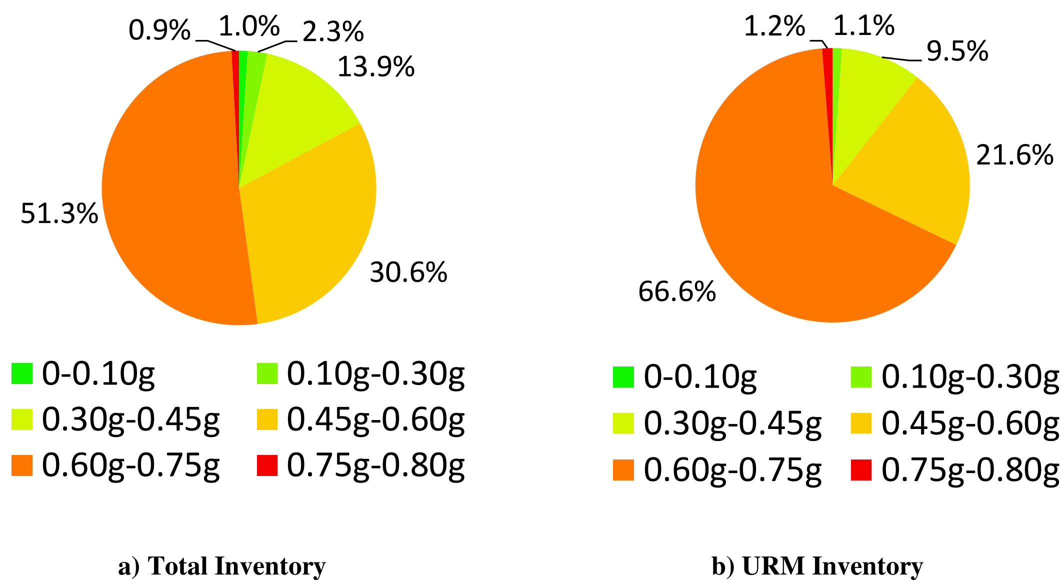


Figure 6-2: Percent Breakdown of Spectral Acceleration of a) Total Inventory, b) URM

6.2.2 Structural Building Damageability

As discussed in section 3.8, structural building damageability (SBD) is computed by aggregating site seismic hazard (SSH) and structural building vulnerability (SBV). Figure 6-4 provides a breakdown of the SBD results for the URM and RC structures within the City of Ottawa building inventory. As expected, the URM buildings show higher damage levels compared to the RC buildings in the database. As seen in Figure 6-4a, when considering URM alone, Over 40% of the buildings are likely to suffer from heavy (I_S^{BD} = 0.6-0.8) to at/or near collapse (I_S^{BD} = 0.8-1.0) damage levels. As shown in Figure 6-4b, when considering the reinforced concrete building stock, the majority of buildings experience damage levels corresponding to none (I_S^{BD} = 0-0.2) and light (I_S^{BD} = 0.2-0.4), with 1.4% of RC buildings experiencing heavy structural damage (I_S^{BD} = 0.6-0.8).

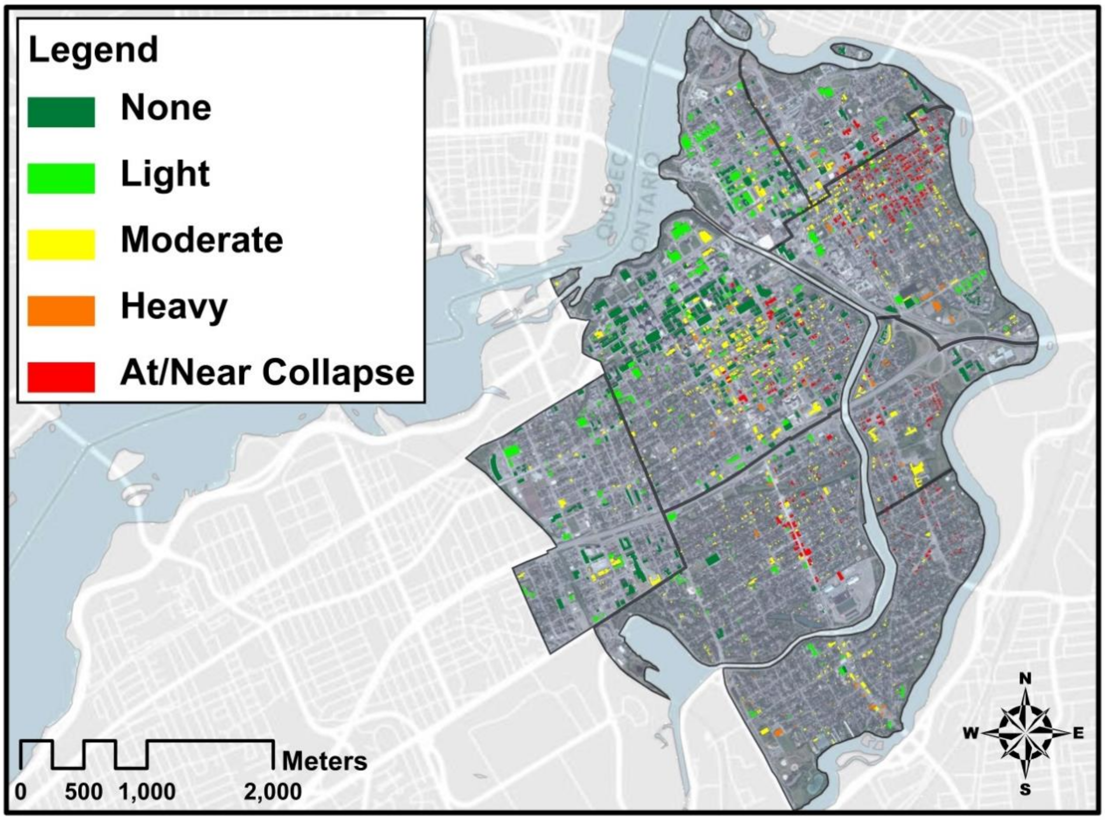


Figure 6-3: Spatial Distribution of Expected Structural Building Damageability of Buildings in Ottawa, Ontario

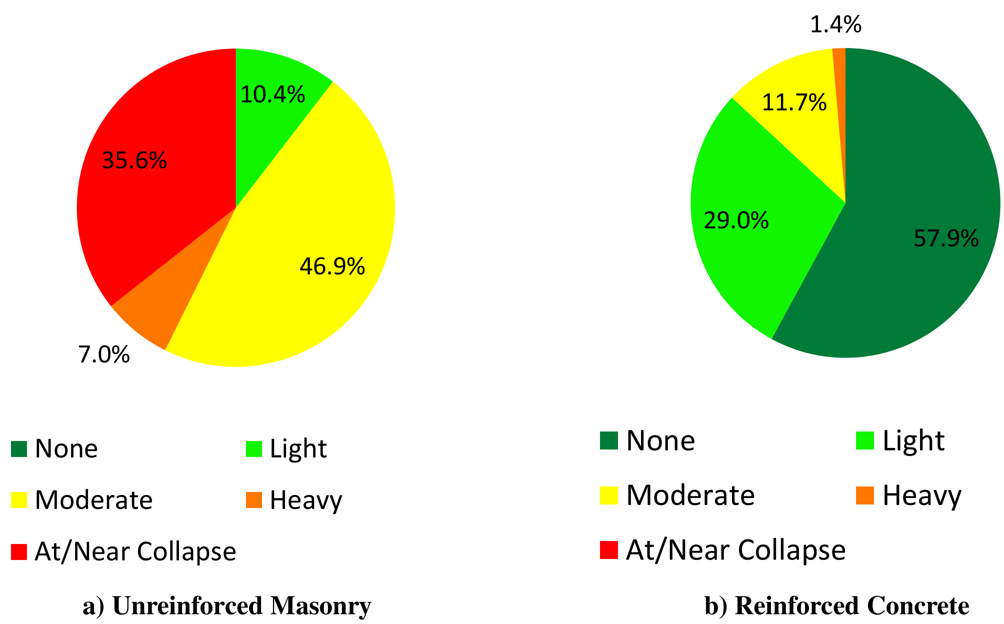


Figure 6-4: Percent Breakdown of Structural Building Damageability: a) URM, b) RC

6.2.3 Structural Risk

As discussed in section 3.9, structural risk (SR) is computed by aggregating structural building damageability (SBD) and building importance/exposure. Figure 6-5 provides a breakdown of the SR results for the URM and RC structures within the City of Ottawa Urban RAT building inventory. As expected, the URM buildings show higher structural risk compared to the RC buildings in the database. When considering URM, Over 40% of the buildings are likely to suffer *critical* to *catastrophic consequences* as seen in Figure 6-6a. A building that may suffer extreme structural damage with the potential to fully collapse poses a major risk to the individuals within the building and any passers-by. With the large number of URM buildings suffering heavy to collapse damage levels, this result is worrying but not surprising. A building with the potential to experience *light* to *moderate* structural damage while being categorized as having low importance may pose *marginal* risk, and this is reflected in Figure 6-6a which shows approximately 57% of buildings in this category .As shown in Figure 6-6b, when considering the reinforced concrete building stock, the majority of buildings show *negligible* to *marginal* risk.

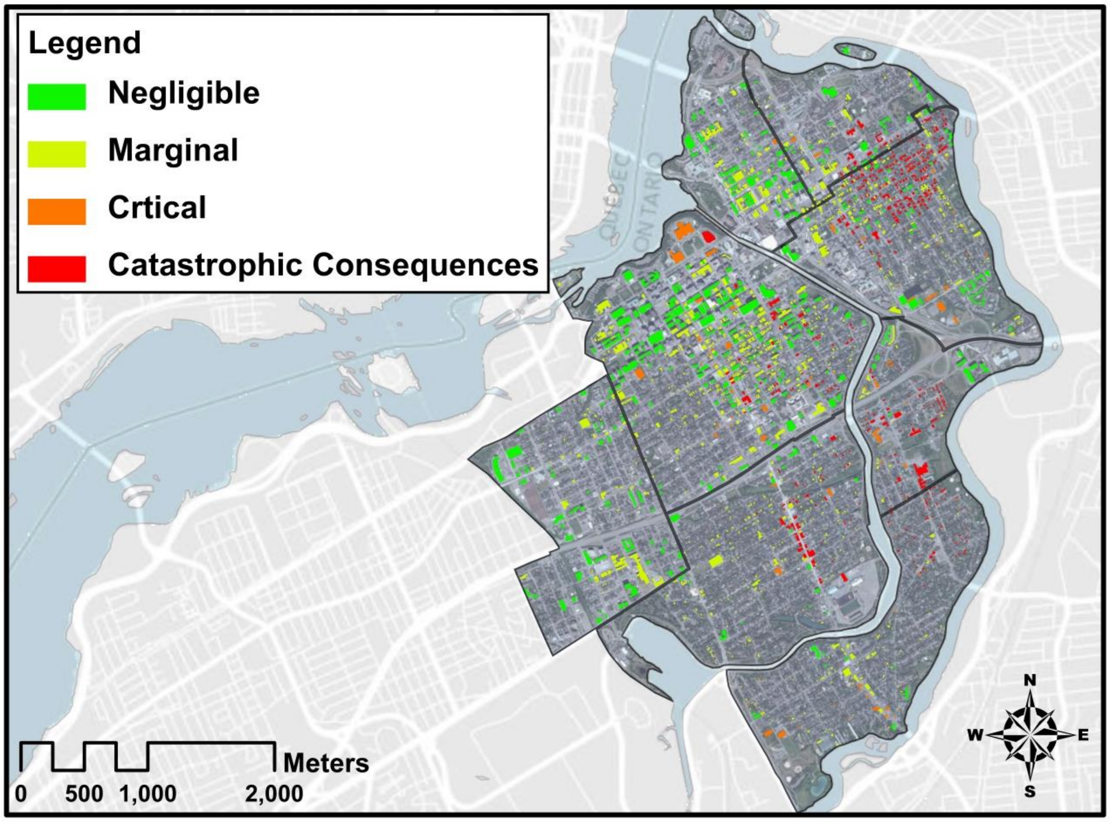
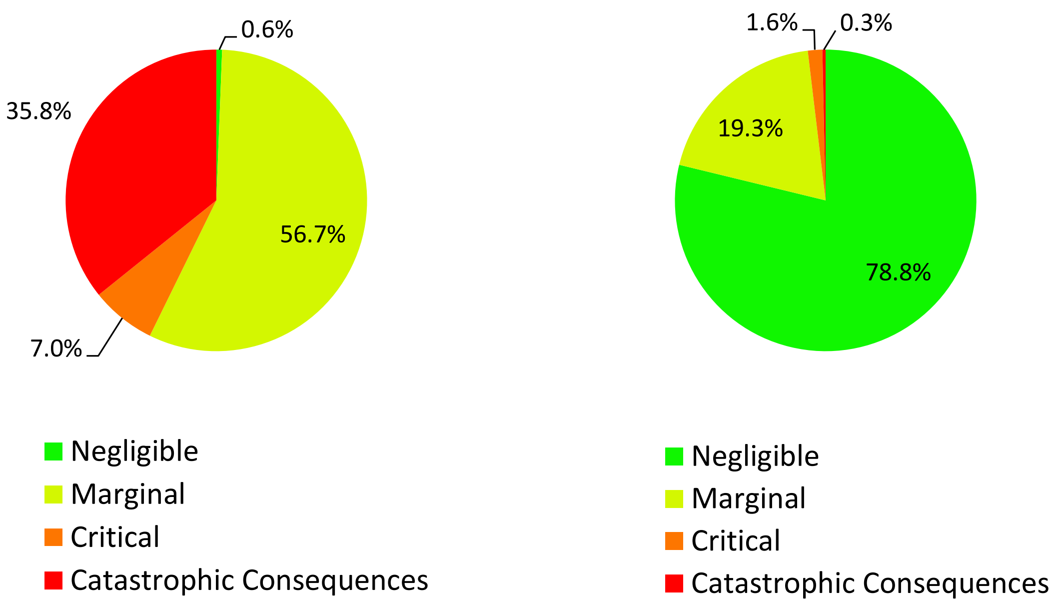


Figure 6-5: Spatial Distribution of Expected Structural Risk of Buildings in Ottawa, Ontario



a) Unreinforced Masonry

b) Reinforced Concrete

Figure 6-6: Percent Breakdown of Structural Risk: a) URM, b) RC

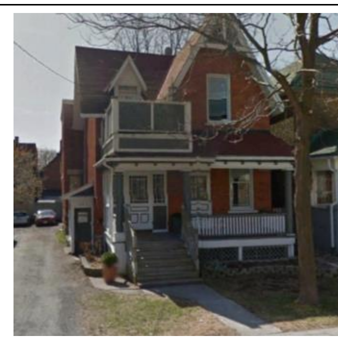
6.3 Case Study

Sixteen URM buildings located in the downtown core were selected (see Figure 6-7) to run a detailed seismic risk assessment using CanRisk. The buildings were verified as being of URM construction based on visual inspection and using historical fire insurance plans (Underwriter's Survey Bureau, 1963). These plans were used by insurance companies in the 1950s to determine the characteristics of building structures that are required to be insured. They include information such as material used to construct the building (stone, concrete, brick, etc.), approximate year of construction (from the date of the fire insurance plans) and number of stories. Half of the buildings assessed are located on Bank Street, a major shopping and business district very similar to the central business district (CBD) located in the Christchurch, New Zealand. Photos of the existing URM buildings are presented in Table 6-1.

Figure 6-7: Location of Selected URM buildings under investigation

Table 6-1: Photos of the URM buildings under investigation

#	Photo	#	Photo
1		2	
3		4	
5		6	
7		8	

#	Photo	#	Photo
9		10	
11		12	
13		14	
15		16	

The input parameters (performance modifiers) used in CanRisk for the buildings under investigation are summarized in Table 6-2. A site visit to each building was conducted to evaluate the building configuration, number of stories, building use, economic impact, vertical irregularity, plan irregularity, construction quality, falling hazards to life and hazards to vital operations. The year of construction is estimated using the fire insurance plans or information from supplementary resources (the internet and GIS information provided from the City of Ottawa). The building area was computed by approximating the building footprint using GIS software and multiplying the computed area by the number of stories. The spectral acceleration Sa(T1) is computed using building location and available microzonation maps for the City of Ottawa (Motazedian et al., 2011), the empirical formulas to determine building period and the corresponding seismic data for Ottawa, Ontario as outlined in the NBCC-2005.

Table 6-2: Summary of Input parameters (performance modifiers) for buildings under investigation

Building Number	Building Address	Configuration	Number of Stories	Soil Type	Spectral Acceleration (g)	Building use	Building Area (m^2)	Economic Impact	Vertical Irregularity	Plan Irregularity	Year of Construction	Construction Quality	Falling Hazards to Life	Hazards to Vital Operations
1	232 Bank St	Row (end)	2	C	0.617	R	450	NG	M	VL	1908	AV	L	VL
2	224-212 Bank St	Row (middle)	3	C	0.521	AP/R	360	NG	H	VL	1904	AV	H	VL
3	219 Bank St	Row (middle)	2	C	0.617	F	619	NG	H	M	1915	PR	M	VL
4	340 Queen St	Stand-alone	2	A	0.461	R	1600	NG	M	L	1950	AV	M	VL
5	427 Bank St	Row (end)	2	D	0.71	U	320	NG	M	VL	1950	AV	M	VL
6	297 Bank St	Row (end)	3	C	0.521	U	1660	NG	VL	M	1950	GD	M	VL
7	399 Bank St	Row (end)	3	D	0.623	AP/R	390	NG	M	VL	1890	GD	M	VL
8	283 Elgin St	Row (end)	4	D	0.542	AP/R	2070	NG	M	H	1950	AV	M	VL
9	368 Bank St	Row (end)	3	C	0.623	AP/R	850	NG	M	VL	1902	AV	M	VL
10	243-249 Bank St	Row (middle)	3	C	0.521	AP/R	390	NG	M	VL	1950	AV	M	VL
11	603 Laurier Av West	Stand-alone	2	A	0.461	AP	443	NG	VL	VL	1935	AV	VL	VL
12	487 Rideau St	Row (end)	2	D	0.71	U	245	NG	H	H	1950	PR	M	L
13	71 Bronson Av	Stand-alone	3	A	0.364	C	660	NG	H	VH	1910	GD	H	VL
14	593 Laurier West	Stand-alone	2	A	0.461	D	660	NG	M	H	1908	GD	M	VL
15	253 Nelson St	Stand-alone	3	D	0.623	AP	482	NG	VL	M	1910	GD	M	VL
16	438 Lisgar St	Stand-alone	2	A	0.524	D	260	NG	VL	VL	1902	GD	VL	VL
VL = Very Low, L = Low, M = Moderate, H =High, VH = Very High, NG = Negligible, GD = Good, AV = Average, PR = Poor, AP = Apartment, D = Dwelling, R = Retail, U = Unknown, C = Church, F = Food														

The results provide the expected performance of the selected URM buildings in the downtown core and foreshadow the possible outcome in the event of a design-level earthquake. While some buildings show *light* structural damage, the majority of the URM buildings experience *moderate* structural damage, with the remainder showing *heavy* or *at/near collapse* structural damage. One building in particular (building 5 in Table 6-3), located on Bank Street, is classified as *at/near collapse* with a structural risk of *critical*. In the event of an earthquake during the daytime, this building may harm the safety of the building occupants or passers-by as it is typically a busy street.

The analysis based on non-structural assessment shows that most URM buildings examined in this case study are expected to have *light* to *moderate* non-structural damage, with a few experiencing *heavy* non-structural damage.

Lastly, when considering overall building risk, the majority of the URM buildings in the case study are found to have *marginal* overall building risk. This is because the URM buildings under investigation are not very significant in terms of economic impact and are typically small in area, housing minimal occupants. However, due to the possibility of collapse for three of the URM buildings in the assessment, the overall building risk results as *critical* due to the potential loss of life as a result of the structural performance of the building. The information provided from the detailed risk assessment presented in this section demonstrates the potential vulnerability of URM buildings in the case of a large magnitude earthquake and may enlighten building owners to consider possible seismic retrofit of structural and/or non-structural elements in order to provide a proper level of building performance.

Table 6-3: Summary of results of buildings under investigation

Building Number	Structural Assessment						Non -Structural Assessment						Overall building risk	
	Structural Vulnerability Index	Structural Vulnerability	Structural Damage Index	Structural Damage	Structural Risk Index	Structural Risk	non-Structural Vulnerability Index	Non-Structural Vulnerability	Non-Structural Damage Index	Non-Structural Damage	Non-Structural Risk Index	Non-Structural Risk	Overall building risk index	Overall building risk
1	0.7	H	0.59	M	0.34	Marg	0.33	L	0.34	LT	0.2	NG	0.24	Marg
2	0.78	H	0.54	M	0.29	Marg	0.69	H	0.54	M	0.29	Marg	0.28	Marg
3	0.78	H	0.66	HV	0.4	Marg	0.61	H	0.62	HV	0.36	Marg	0.36	Marg
4	0.75	H	0.5	M	0.24	Marg	0.44	M	0.43	M	0.24	Marg	0.2	NG
5	0.67	H	0.81	Col	0.57	Crit	0.44	M	0.43	M	0.24	Marg	0.41	Crit
6	0.49	M	0.33	LT	0.19	NG	0.46	M	0.45	M	0.24	Marg	0.15	NG
7	0.7	H	0.59	M	0.34	Marg	0.46	M	0.44	M	0.24	Marg	0.24	Marg
8	0.8	H	0.57	M	0.32	Marg	0.62	H	0.58	M	0.33	Marg	0.32	Marg
9	0.7	H	0.59	M	0.34	Marg	0.46	M	0.44	M	0.24	Marg	0.24	Marg
10	0.69	H	0.47	M	0.24	Marg	0.44	M	0.43	M	0.24	Marg	0.2	NG
11	0.63	H	0.38	LT	0.22	Marg	0.33	L	0.35	LT	0.2	NG	0.18	NG
12	0.81	VH	0.92	Col	0.64	CC	0.7	H	0.74	HV	0.35	Marg	0.59	Crit
13	0.81	VH	0.92	Col	0.64	CC	0.7	H	0.74	HV	0.35	Marg	0.59	Crit
14	0.82	VH	0.58	M	0.33	Marg	0.63	H	0.43	M	0.24	Marg	0.24	Marg
15	0.5	M	0.54	M	0.29	Marg	0.49	M	0.48	M	0.24	Marg	0.22	Marg
16	0.63	H	0.38	LT	0.22	Marg	0.33	L	0.34	LT	0.2	NG	0.18	NG
VL = Very Low, L = Low, M = Moderate, H =High, VH = Very High, NG = Negligible, Marg = Marginal, Crit = Critical, CC = Catastrophic Consequences, LT = Light, HV = Heavy, Col = At/Near Collapse,														

Chapter 7
Summary, Conclusion and Future Recommendations

7.1 Summary

The performance of unreinforced masonry structures in past earthquakes clearly demonstrates the seismic vulnerability of this construction type. In Canada, many densely populated cities have large inventories of unreinforced masonry structures. With many Canadians living in areas of high or moderate seismicity, it is essential to understand the potential hazards posed by vulnerable URM buildings. Many existing URM structures have not been retrofitted and remain at risk in the event of a large magnitude earthquake. There is therefore a need to identify vulnerable structures and develop tools for assessing the seismic vulnerability of masonry structures in Canada.

Due to large inventories of URM buildings and the hazard associated with this structural type, it is essential to include the evaluation of URM buildings when conducting seismic risk assessments. Seismic risk assessments provide knowledge to support effective actions by decision makers that can reduce potential damage to populated urban communities. In the case of seismically deficient URM buildings, information gathered from risk assessments can provide insight on potential mitigation techniques (retrofit, demolition, etc.).

A risk-based seismic assessment tool (CanRisk) is proposed to assess the seismic vulnerability of existing unreinforced masonry and reinforced concrete buildings. The tool exploits the use of fuzzy logic, a soft computing technique, to capture the vagueness and uncertainty within the evaluation of the performance of a given building. CanRisk was developed to determine the overall building's risk in terms of structural and non-structural assessments. The model was tested using a sensitivity analysis and validated using information following the 2011 Christchurch Earthquake in New Zealand.

In order to conduct seismic risk assessments, a general building inventory and its spatial distribution and variability is required for earthquake loss estimations. The thesis presents the application of Urban Rapid Assessment Tool (Urban RAT), a tool designed for the rapid collection of building data in urban centres. The Geographic Information System (GIS) based assessment tool allows for intense data collection and revolutionizes the traditional sidewalk survey approach to collecting building data.

A practical investigation on the seismic performance of structures was conducted to assess buildings found in populated urban centers. The predominant structure type under investigation was unreinforced masonry, known to be the most hazardous form of construction in the event of an earthquake. Many urban centres contain a large building stock, therefore software and hardware tools that can expedite data collection and the evaluation of existing structures are fundamental to timely seismic risk mitigation decisions. CanRisk provides earthquake loss estimations by using building specific information to establish the performance and risk of a given building. In addition, a tool that revolutionizes the traditional side-walk survey of collecting building data is presented. The Urban RAT suite can better equip regions to mitigate and prepare for, respond to, and recover from natural disasters including earthquakes for emergency management purposes. The advancements in data processing and GIS has provided the foundation for the development of comprehensive loss estimation programs such as CanRisk that can better serve decision makers in Canada. The City of Ottawa, an area of moderately high seismic risk, has a population of almost one million people and it is essential to evaluate distribution of seismic risk across the city, especially within heavily populated and historical regions such as the downtown core. The preliminary results of the use of CanRisk and the Urban RAT were used to provide building information and assess the performance of a large stock of URM and RC buildings in the City of Ottawa.

7.2 Conclusions

This thesis has highlighted the vulnerability of URM buildings in the event of an earthquake. The development of tools in order to conduct seismic risk assessments are proposed and applied to the City of Ottawa. From this study, the following particular points can be concluded:

- URM buildings pose significant seismic risk. Irregular URM buildings show higher vulnerability. Single, stand-alone URM buildings develop higher seismic damage in comparison with row buildings. In row buildings, end units suffer more damage than inside units.
- Non-ductile buildings, including older non-ductile reinforced concrete buildings, pose higher seismic vulnerability. Seismic vulnerability of reinforced concrete buildings has reduced over the years due to improved provisions of codes and standards. The improvements can be grouped in four eras in terms of year of construction; pre-1975; 1975 to 1985; 1985 to 2005; post 2005.
- Structural vulnerability can be assessed by considering structural deficiencies associated with irregularities as parameters that increase seismic demands, and structural force and deformation capacities as reflected by year of construction and the quality of construction.
- Non-structural vulnerability can be assessed by considering seismic deformation demands in the building, potentials for falling hazards and impact on vital operations through loss of operational and functional components.
- The hierarchical model developed by Tesfamariam and Saatcioglu (2008), as modified in the current investigation to incorporate unreinforced masonry buildings and non-structural components, provides good estimates of building damageability and seismic risk as shown by a sensitivity analysis and verification against observed building damage.
- The seismic risk assessment tool, CanRisk that was improved and expanded in the current investigation provides good estimates of building damageability and seismic risk for URM and reinforced concrete buildings. CanRisk is a seismic risk assessment tool for URM and RC buildings that integrates site seismic hazard, building vulnerability and building importance to provide building damage and risk estimations.
- Uncertainty and ambiguity involved in the seismic evaluation of a building is captured through hierarchical fuzzy rule-based modeling.
- The sensitivity analysis conducted using CanRisk provide results that conform to current knowledge, experience and reconnaissance observations available in the literature.

- Verification of the seismic risk assessment tool (CanRisk) against selected URM buildings damaged during the 2011 Christchurch Earthquake in New Zealand indicate good correlations between the results of the assessment tool and observed damage.
- Urban RAT electronic tool, developed on a synchronized ArcGIS-Google-Android platform allowing both in-lab/virtual assessments and in-field/on-site assessments, can be used effectively for collecting building data and inventory to perform seismic risk assessment.
- The ability to perform in-lab/virtual site assessments as well as the auto-fill function embedded in Urban RAT optimizes time and efficiency of data collection;
- The inclusion of engineering parameters, based on FEMA 154 (ATC 2002) and FEMA 310 (ASCE 1998), within the Urban RAT suite provides data which can be used in loss estimation programs, and the potential to build a very well-developed building inventory across a large urban area;
- Information and data collected from Urban RAT as presented in this study can be utilized in earthquake loss estimation models such as CanRisk and HAZUS to provide loss estimations which can ultimately be used in disaster management and mitigation programs.
- Within the building inventory for the City of Ottawa, 5.8% of total building stock are classified as engineered building while the remainder are non-engineered buildings built from prescriptive methods (URM and wood-frame buildings). This is a result of a large inventory of residential single family dwellings included in the assessment;
- Roughly 35% of the total building inventory and 55% of the URM building stock in Ottawa is constructed on soft/stiff soil profiles (Class D & E) that include an increased ground shaking characteristics during the event of an earthquake;
- Approximately 1.0% of the total building stock in Ottawa is categorized as high importance or post-disaster buildings. While 3% of URM buildings assessed fall in the post-disaster and high-importance categories;
- Nearly 75% of buildings and over 80% of URM buildings in the downtown core of Ottawa were built prior to 1940 (prior to the development of seismic design criteria), and thus need to be evaluated to ensure a satisfactory degree of safety in the event of a large magnitude earthquake;

- 8.6% of the total buildings and 35.2% of URM buildings in Ottawa are tagged as including an irregular structural configuration. Building irregularity is an important parameter that must be identified to assess the performance of buildings during earthquakes;
- 1,465 URM and 580 RC buildings were evaluated in Ottawa for structural building damage and risk in the event of a design level earthquake. Results demonstrate 53% of the total inventory and ~68% of URM buildings may experience a spectral acceleration of 0.6g (due to most inventories sitting in the short period range). In terms of building damage, 35.6% of URM are likely to suffer “at/near collapse” while the majority of RC buildings see little to no damage. Finally, 43% of URM and ~2.0% of RC buildings show critical to catastrophic consequences as a result of building importance and structural building damageability.
- A detailed risk assessment was conducted (structural and non-structural) of 16 URM buildings found of the downtown core in the City of Ottawa. Results show 3 URM buildings displaying critical overall building risk due to the potential structural collapse of these buildings.

7.3 Future recommendations

Based on the research described in this thesis, the following areas require further study:

- The inclusion of additional structural systems (such as wood/timber and steel structures) for the assessment of seismic risk.
- The inclusion of a social consequences/casualties module to predict injuries and casualties related to seismic risk.
- The inclusion of an economic consequences module to predict monetary losses in the evaluation of a building.
- Further calibration of the CanRisk model to earthquake reconnaissance reports to provide further fine tuning of the tool.
- The integration of CanRisk into a GIS or Google Map platform to provide disaster management an effective way of conducting seismic risk evaluations.
- Implementation of CanRisk into an open-source programming language such as the .net framework for ease of distribution

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Appendix A

The following appendix provides instructions relevant to the installation of *CanRisk* program followed by a tutorial illustrating how to utilize the software in order to conduct a seismic risk assessment of a building

Installation

In order to utilize the *CanRisk* program, users must determine the system type of their computer; this can be done by going to start → computer → system properties. Users running a 32-bit system type should install the 32-bit package of *CanRisk* while users running a 64-bit system should install the 64-bit package. To install, begin by double clicking the CanRisk_pkg.exe which will execute and extract all files necessary. When prompted install the MCR (Matlab Compiler Runtime) to complete the installation (this need to only be done once). Finally, open the CanRisk.exe file which will open a standalone ***CanRisk*** application for the user to begin conducting seismic risk assessments.

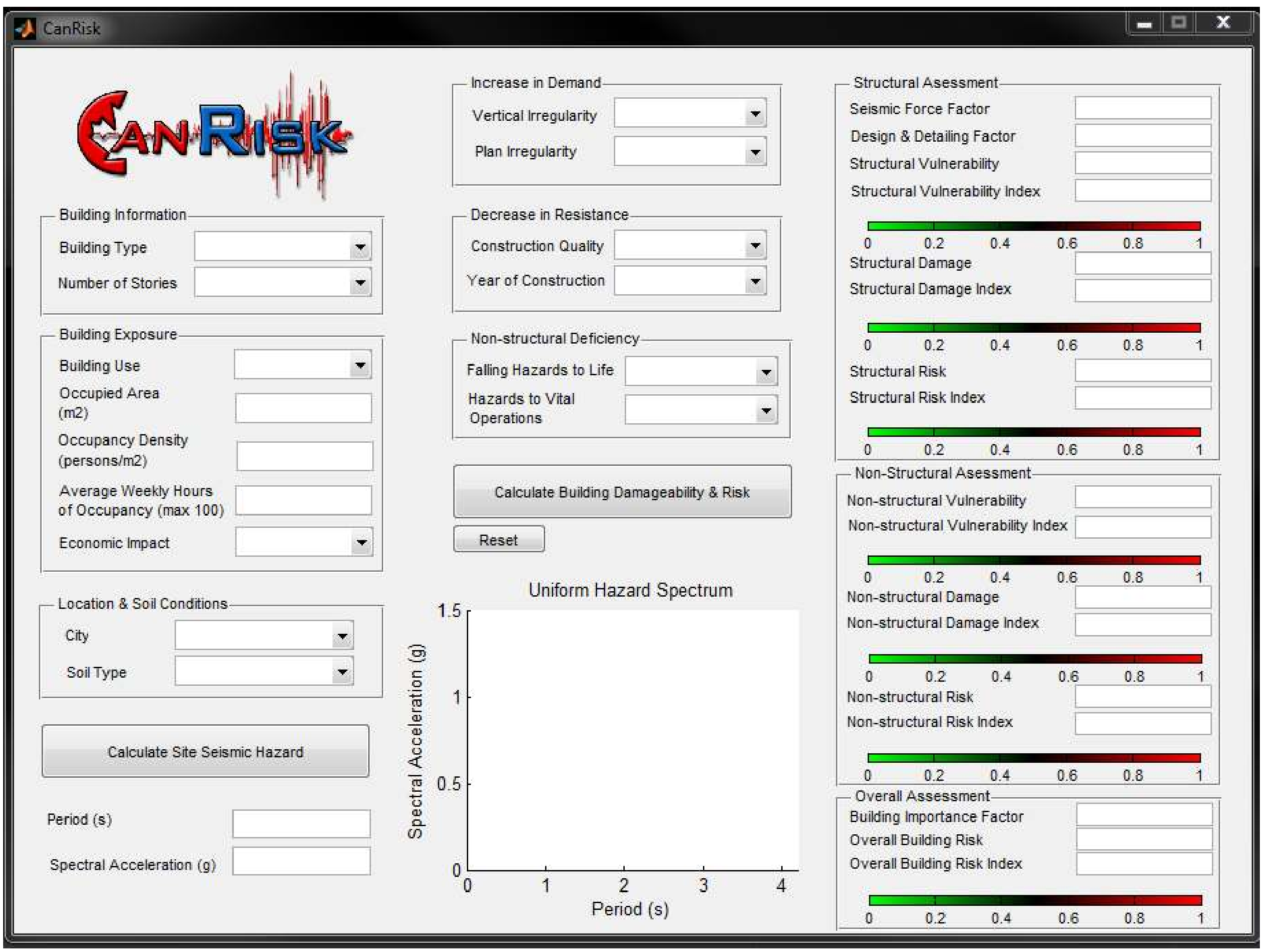
Example:

Consider an unreinforced masonry (URM) load-bearing building located in the city of Ottawa, Ontario. The URM building is 3 stories and is a stand-alone URM building. The soil type is class "C" and the year of construction of the building is dated 1955. From the side-walk survey, the construction quality is "Poor" from extreme deterioration of the bricks and mortar and the vertical irregularity and plan irregularity of the building is assessed as "Very Low" and "Moderate", respectively. The URM building is of original construction and appears to be constructed with parapets protruding from the roofline ("High" falling hazards). No hazards to the operation of the building are present ("Very Low" hazards to vital operations). Assume that the building is residential, a “Negligible” economic and a building area of approximately 250m^2. Assess the seismic risk level of the building.

The following demonstrates a step by step procedure of *CanRisk* in order to illustrate the seismic risk assessment of the aforementioned building.

Step 1: Start *CanRisk*

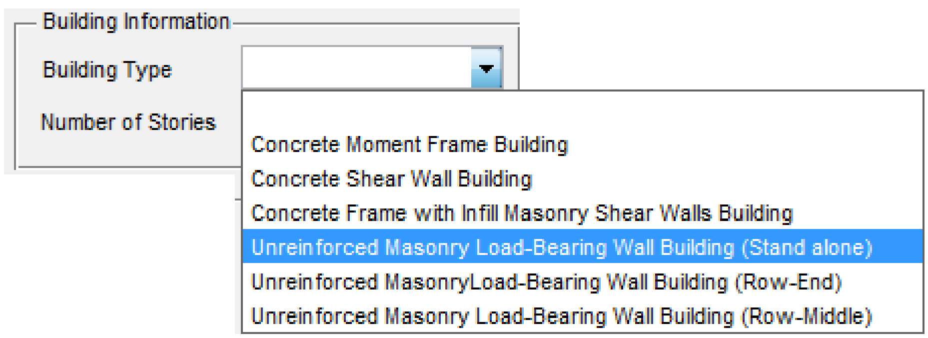
Start the ***CanRisk*** program by double clicking the CanRisk.exe file. Once the program is opened, the input data is inserted manually by the user by selecting various options of the graphical user interface (GUI) of ***CanRisk*** as seen below.



Step 2: Enter Building Information

The **Building Information** panel pertains to the building structural type under investigation and the number of stories of that building.

- For **Building Type**, select **URM: Stand-alone Building**
- For **Number of Stories**, select **3**

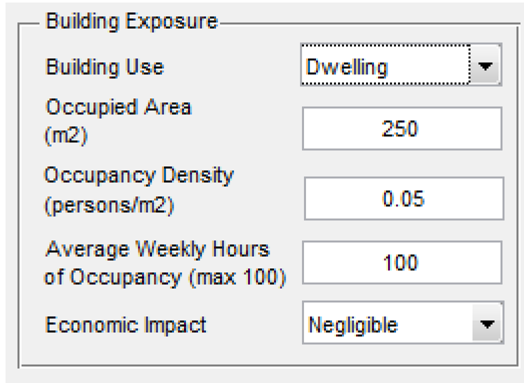


Step 3: Enter Building Exposure

The **Building Exposure** panel provides information relevant to compute the consequence of failure of the building under investigation

- For **Building Use**, select **Dwelling**
- For **Occupied Area**, enter **250**

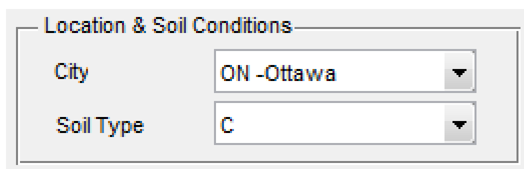
Default values for the **Occupied Density** and **Average Weekly Hours of Occupancy** have been formulated for each **Building Use**, but the user has the capability of enter values for better approximations of building importance. The default values are used in the following assessment.



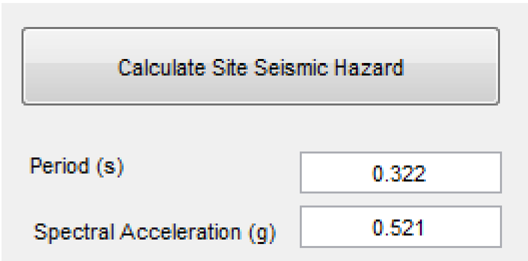
Step 4: Enter Building Location and Soil Conditions

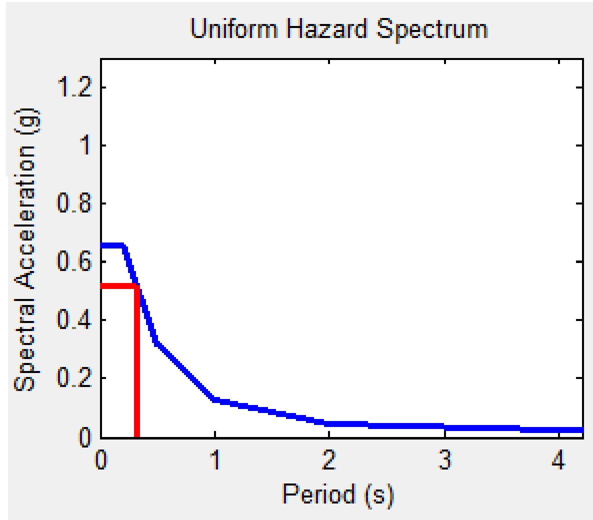
The **Location & Soil Conditions** panel provides information necessary to construct the Uniform Hazard Spectrum (UHS) in order to obtain the period (T_1) spectral acceleration (Sa) of a given structural type which will provide an indication of site seismic hazard (SSH).

- For **Location**, select **ON -Ottawa**
- For **Soil Type**, enter **C**



Once selected, click the **Calculate Site Seismic Hazard** button to determine the period, spectral acceleration and display the UHS for the building.



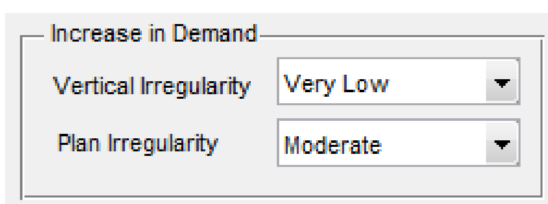


Therefore, the URM building has a period of 0.322 seconds and a spectral acceleration of 0.521g. The value obtained for spectral acceleration will be used in order to quantify SSH.

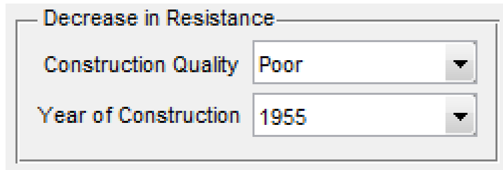
Step 5: Enter Building Characteristics

The **Increase in Demand**, **Decrease in Resistance** and **Non-structural Deficiency** panels provide information in order to quantify the structural and non-structural building vulnerability. The information selected pertains to the inherent building characteristics and reflects the capacity/demand as well as the non-structural deficiency that can affect the overall vulnerability of a structure.

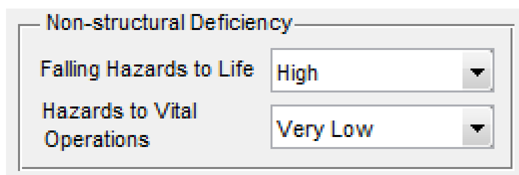
- For **Vertical Irregularity**, select **Very Low**
- For **Plan Irregularity**, select **Moderate**



- For **Construction Quality**, select **Poor**
- For **Year of Construction**, select **1955**



- For **Falling Hazards to Life**, select **High**
- For **Hazards to Vital Operations**, select **Very Low**



Once all the input parameters have been selected, the user clicks the **Calculate Building Damageability** & Risk button in order to obtain the results of the assessment.

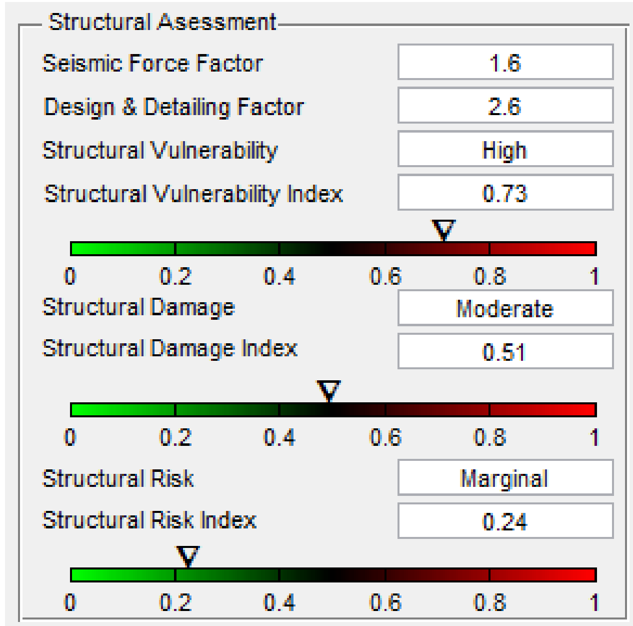
Step 6: Risk Assessment Results

Once computed, the results are presented in three different panels: **Structural Assessment**, **Non-Structural Assessment** and **Overall Assessment**

Structural Assessment

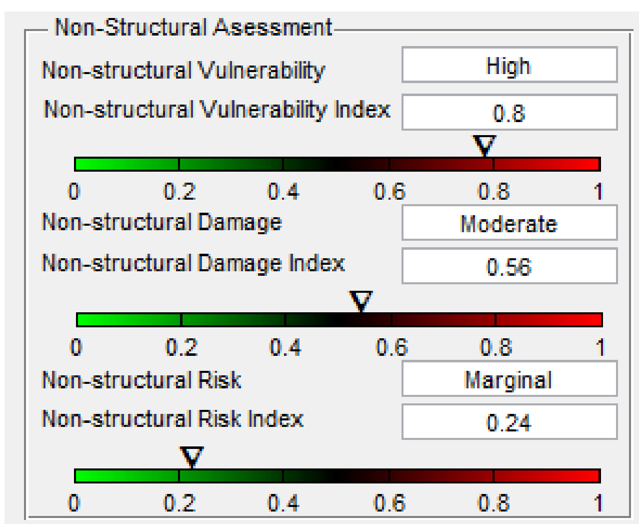
The **Structural Assessment** panel displays the Seismic Force Factor (SFF), the Design & Detailing Factor (DDF), Structural Vulnerability (SV), Structural Damageability (SD) and the Structural Vulnerability Index (SVI). Thus, the results of the example above are:

- **High** Structural Vulnerability
- **Moderate** Structural Damage (function of SSH and SV)
- **Marginal** Structural Risk (function of SD and Building Exposure)



The **Non-structural Assessment** panel displays the Non-structural Vulnerability (NSV), Non-structural Damageability (NSD) and the Non-structural Vulnerability Index (NSVI). Thus, the results of the example above are:

- **High** Non-structural Vulnerability
- **Moderate** Non-structural Damage (function of SSH and NSV)
- **Marginal** Non-structural Risk (function of NSD and Building Exposure)



Finally, the Structural Risk and Non-structural Risk are aggregated to give an Overall Building Risk level and index. Thus, the results of the example are:

- **Marginal** Overall Building Risk