

# **Seismic Fragility Analysis of Reinforced Concrete Shear Wall Buildings in Canada**

By

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Thesis submitted to the

**Faculty of Graduate and Postdoctoral Studies**

in partial fulfilment of the requirements for the

**Doctorate in Philosophy Degree in Civil Engineering**



uOttawa

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## **Abstract**

Damage observed after previous earthquakes indicates that a large number of existing buildings are vulnerable to seismic hazard. This research intends to assess seismic vulnerability of regular and irregular shear wall buildings in Canada, having different heights and different levels of seismic design and detailing. As seismic hazard is a probabilistic event, a probabilistic methodology has been adopted to assess the seismic vulnerability of the shear wall buildings. The proposed research encompasses a comprehensive fragility analysis for seismic vulnerability of shear wall buildings in Canada. The first phase of the investigation involves shear wall buildings with different heights (hence different structural periods), designed based on the 2010 National Building Code of Canada. The second phase involves shear wall buildings designed prior to 1975, representing pre-modern seismic code era. The third phase involves the evaluation of pre-1975 shear wall buildings with irregularities.

3-Dimensional simulations of the buildings were constructed by defining nonlinear modelling for shear wall and frame elements. These models were subjected to dynamic time history analyses conducted using Perform 3D software. Two sets of twenty earthquake records, compatible with western and eastern Canadian seismicity, were selected for this purpose. Spectral acceleration and peak ground acceleration were chosen as seismic intensity parameters and the first storey drift was selected as the engineering demand parameter which was further refined for irregular cases. The earthquake records were scaled to capture the structural behaviour under different levels of seismic excitations known as Incremental Dynamic Analysis. The resulting IDA curves were used as the input for seismic fragility analysis. Fragility curves were derived as probabilistic tools to assess seismic vulnerability of the buildings. These curves depict probability of exceeding immediate occupancy, life safety and collapse prevention limit states under different levels of seismic intensity.

## **Acknowledgements**

I would like to express my deepest gratitude to my supervisor, Professor Saatcioglu, for his unrelenting support, constructive guidance and encouragement throughout my graduate studies.

Also, I would like to express my infinite appreciation to my parents Forouz and Manoucher and my lovely sister Niloofar, for their unconditional love and support throughout my academic journey.

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# Chapter 1. Introduction

## 1.1. Research Needs

Structural damage observed after previous earthquakes revealed that a large number of existing buildings are vulnerable to strong ground motions. These observations have resulted in improved seismic design provisions in modern building codes. Depending on the year of construction, the level of seismic design and detailing, and the effects of irregularities in buildings, a large inventory of building infrastructure in Canada remains seismically vulnerable. Seismic vulnerability can best be assessed using a probabilistic approach consistent with the probabilistic nature of seismic hazards.

Seismic vulnerability assessment of an individual building, either deterministically using inelastic dynamic analysis or probabilistically through fragility analysis, is a time consuming task. It is not a common practice to derive fragility curves for buildings case by case, except for important post-disaster buildings or major buildings with potentials for serious vulnerability and consequences of failure. Hence, it is necessary to classify buildings into representative groups and assess the vulnerability of a specific building in a group by utilizing available tools that have been developed for the appropriate group.

Classifying building types into appropriate categories, and developing fragility curves for each category provides useful seismic risk assessment tools.

Most of the existing buildings in Canada are residential buildings designed during the pre-1975 era, prior to the development of modern seismic codes. These buildings did not benefit from ductile seismic design and detailing practices of current building codes. Others, designed since the 1975 National Building Code of Canada (NRCC 1975) have improved seismic performance, but their performance may need to be assessed relative to pre-selected performance objectives, as well. Furthermore, the performance of buildings in the latter category may need to be assessed under increased seismic hazards, as seismic hazard values have changed since 1975. Therefore, the building inventory in Canada was divided into three categories; i) regular buildings designed using post-1975 NBCC, ii) regular buildings designed using pre-1975 NBCC, iii) irregular buildings designed using pre-1975 NBCC. The treatment of irregularities in recent codes has been improved; therefore the vulnerability of irregular buildings designed to the recent editions of NBCC is not included within the scope of the current research program. Seismic hazards corresponding to uniform hazard spectra have been adopted as the reference seismic hazard values for the purpose of developing fragility curves. The current research project forms part of a comprehensive research program on seismic vulnerability assessment of reinforced concrete and masonry buildings in Canada currently underway at the University of Ottawa, and it focuses on reinforced concrete shear wall buildings.

Seismic vulnerability of buildings varies based on the seismicity of the region and associated design and detailing practices employed. Canadian seismicity can be viewed in two categories; seismicity of western Canada, and seismicity of eastern Canada. Earthquakes in the west are associated with active faults. They may occur along the faults in the offshore region, within the subduction zone in the Pacific Ocean near the British Columbia Coast, or inland within the continental crust. These earthquakes are

sources of high seismic hazard values, generally viewed as “strong earthquakes.” Western earthquakes generally exhibit low frequency content and they dissipate quickly with distance. Eastern Canada is located in a stable region on the North American Plate, away from active fault lines. However, damaging earthquakes have previously occurred in this region. The causes of earthquakes in eastern Canada are not well understood. They can be explained by the regional stress fields, with the earthquakes concentrated in regions of crustal weakness (EarthquakesCanada 2016). Eastern Canadian earthquakes are generally viewed as “medium intensity earthquakes”. They have higher frequency content, and attenuate more slowly with distance. Because of the two distinctly different seismic characteristics of Canada, two cities were selected in the current investigation for the development of seismic fragility curves; Vancouver in the west and Ottawa in the east. Vancouver is a city with high seismic hazard and high concentration of population, potentially having vulnerable buildings and high seismic risk. The major metropolitan centers in eastern Canada with medium level of seismicity include Ottawa and Montreal. These two cities have similar uniform hazard spectra in the 2010 NBCC, though some differences have been introduced in the 2015 NBCC.

In Ottawa, around 90% of the buildings are residential, 88% being designed prior to 1970 (El Sabbagh 2014). In Western Canada, more than 40% of those are designed prior to 1970, with 50% of the residential units being single-family homes and 34% being apartment units (Ventura *et al.* 2005). Three different building heights; 2-Storey, 5-Storey and 10-Storey buildings, were selected as representative buildings for the purpose of generating fragility curves for reinforced concrete shear wall buildings.

## 1.2. Objective

The objective of this research project is to develop seismic vulnerability assessment tools in the form of fragility curves for representative classes of reinforced concrete shear wall buildings in Canada, categorized into three groups; i) regular buildings designed after the

enactment of modern building codes (conforming to the 2010 NBCC), ii) regular buildings designed in the pre-code era (conforming to the 1970 NBCC), and iii) irregular buildings designed in the pre-code era. The objective also includes assessment of reinforced concrete shear wall buildings in Canada by using the results of the fragility analysis.

### 1.3. Scope

The following steps form the scope of the current investigation:

- *Review of previous literature; and seismic codes and standards:* The previous literature on performance and seismic fragility analysis of reinforced concrete buildings, as well as the evolution of codes and standards was conducted in three steps.
  - Literature on code conforming and pre-code buildings with regular structural layouts.
  - Literature on irregular buildings.
  - Evolution of the seismic provisions of the National Building Code of Canada (NBCC) and the seismic design and detailing requirements of CSA Standard A23.3.
- *Selection of buildings:* Reinforced concrete shear-wall buildings with three building heights (2-storey, 5-storey and 10-storey), were selected with Vancouver and Ottawa as their locations, representing buildings subjected to western and eastern Canadian seismicity, respectively.
- *Categorization of buildings based on year of construction:* The shear wall buildings were categorized into three groups:

- *Post-1975 Regular Buildings.*
- *Pre-1975 Regular Buildings.*
- *Pre-1975 Buildings with Irregularities.*
- *Selection and verification of computer software for dynamic time history analysis:* PERFORM-3D was selected for non-linear response time history analysis. The software was verified against available large-scale shear wall tests.
- *Modelling of buildings for PERFORM-3D analysis:* Three dimensional building models were generated for use with PERFORM-3D. The models consisted of beam and column elements with potential plastic hinges at their ends, and a strip model simulating hysteretic characteristics of shear walls.
- *Selection of site-specific earthquake records compatible with the Uniform Hazard Spectra (UHS) specified in the 2010 NBCC for use in dynamic analysis:* A total of 20 synthetic earthquake records were selected for each location in western and eastern Canada for use in dynamic analysis.
- *Incremental Dynamic Analysis (IDA):* IDA was conducted to generate data for fragility analysis under incrementally increasing seismic intensity.
- *Selection of target building performance objectives:* Building performance levels were selected as i) Immediate Occupancy, ii) Life Safety, and iii) Collapse. These target building performance levels were quantified based on first-storey drift ratio as the damage indicator.
- *Statistical analysis of IDA results to develop fragility curves:* Fragility curves were developed for three sets of buildings; i) post-1975 buildings with regular structural layouts, ii) pre-1975 buildings with regular structural layout, and iii) irregular

buildings. The curves were generated using spectral acceleration and peak ground acceleration (PGA) as earthquake intensity measures.

- *Seismic vulnerability assessment:* Using the fragility curves developed, an assessment of pre- and post-1975 shear wall buildings with regular structural layouts was made. The effects of plan and elevation irregularities were demonstrated relative to pre-1975 buildings.

#### 1.4. Original Contributions

The research carried out in the thesis resulted in a number of original contributions in the area of seismic assessment of shear wall buildings in Canada. These can be grouped in three categories:

- Development of seismic fragility curves for 30 categories of Canadian shear wall buildings covering a wide range of building periods, building heights, and irregularities, while considering year of construction and differences in eastern and western Canadian seismicity. The curves provide convenient tools for regional seismic assessment of shear wall buildings in Canada, a set of probabilistic tools that have not been developed before in the comprehensive manner in which they were developed.
- Generation and application of detailed nonlinear numerical models of shear wall buildings for nonlinear time history analysis. This includes fiber models for flexural behavior and nonlinear sectional model for shear, resulting in more realistic displacements in the inelastic range of deformations. The models also enable better simulation of the possibility of developing shear distress prior to flexural failure.
- The treatment of common irregularity types in existing Canadian shear wall buildings and the development of fragility curves for irregular buildings. This has not been done earlier in the systematic manner in which it has been covered in the current research program.

## 1.5. Presentation of the Thesis

This thesis is organized primarily in paper format. It consists of 7 Chapters, as indicated below. Chapters 4, 5 and 6 include papers either accepted or submitted for publication. These Chapters consist of the reproduction of the papers. Chapter 3 is based primarily on another paper, but follows a different format, focusing on model development and analytical techniques. Because of the paper format adopted, some minor duplication of concepts in various Chapters could not be avoided.

- **Chapter 1** introduces research needs, objectives and scope with original contributions highlighted.
- **Chapter 2** provides a literature review of previous research projects, as well as an overall review of seismic building code and standard development in Canada over the years.
- **Chapter 3** presents “Analytical modelling of reinforced concrete shear wall buildings for dynamic analysis” which forms part of a technical paper that discusses computer software PERFORM-3D employed for dynamic analysis, its validation against available experimental data, and analytical modelling of buildings for dynamic inelastic response history analysis. The following is the citation of the paper that has been accepted for publication.

*Rafie Nazari and Saatcioglu. 2017. Seismic performance assessment of concrete shear wall buildings through fragility analysis. Accepted for publication in the Journal of Building Engineering, Elsevier.*

- **Chapter 4** presents the development of “Fragility curves for Canadian shear wall buildings conforming ductile design requirements” in the form of a technical paper, which is cited below.

*Rafie Nazari and Saatcioglu. 2017. Fragility curves for Canadian shear wall buildings conforming ductile requirements. Provisionally accepted for publication in the Canadian Journal of Civil Engineering.*

- **Chapter 5** includes “Fragility curves developed for non-ductile pre-1975 regular buildings” in the form of a technical paper, which is cited below.

*Rafie Nazari and Saatcioglu. 2017. Seismic fragility analysis of non-ductile shear wall buildings in Canada. Under review for publication in the Canadian Journal of Civil Engineering.*

- **Chapter 6** presents “Seismic performance assessment of shear wall buildings with plan and elevation irregularities” in the form of a technical paper, which is cited below.

*Rafie Nazari and Saatcioglu. 2017. Seismic fragility analysis of shear wall buildings with plan and elevation irregularities. Under preparation for publication in the Earthquake Spectra, the Journal of Earthquake Engineering Research Institute.*

- **Chapter 7** provides summary, conclusion and recommendations for future research.
- **Appendix A** discusses “Site specific record selection and fragility analysis” in the form of a conference paper, which is cited below.

*Rafie Nazari and Saatcioglu. 2017. Site specific record selection and fragility analysis. Proceedings of the Annual Conference of the Canadian Society for Civil Engineering (CSCE), Vancouver.*

- **Appendix B** demonstrates the application of fragility curves to a multi-storey shear wall building designed for Montreal as an illustrative example.

The following papers, also published from the research carried out as part of the current PhD thesis, but not included in the thesis are as follows:

- *Rafie Nazari and Saatcioglu. 2015. Seismic performance assessment of concrete shear wall buildings through fragility analysis. Proceedings of the 11th Canadian Conference on Earthquake Engineering (11CCEE), Victoria.*
- *Rafie Nazari and Saatcioglu. 2017. Seismic vulnerability assessment of Canadian reinforced concrete shear wall buildings designed in pre-modern seismic code era. Proceedings of the Annual Conference of the Canadian Society for Civil Engineering (CSCE), Vancouver.*

# Chapter 2. Literature Review

## 2.1. Previous Studies on Seismic Performance of Shear Wall Buildings

There has been limited research on seismic vulnerability assessment of shear wall buildings. A review of previous research is presented in the following paragraphs.

Ji *et al.* (2007, 2009) developed analytical framework for seismic fragility assessment of reinforced-concrete high-rise buildings using a simple lumped-parameter model representative of the complex high-rise building system in the ZEUS–NL, which is a state-of-the-art 3D static and dynamic analysis platform. The model consisted of beam elements, rigid bars and non-linear springs. They applied generic algorithms to choose parameters of the model, which were selected and implemented for an existing high-rise structure with dual core walls and a reinforced concrete frame. The behaviour of the wall was considered in flexure, shear and axial load while also considering their interactions. The spring stiffness values for the outer frame were established using ZEUS–NL and the lumped wall model was established by using VecTor2, non-linear continuum analysis software. The suggested simplified model resulted in significant reductions in the analysis time, and can be applied to fragility analysis of complex high-rise structures.

Koduru and Haukaas (2009) modeled a 15 storey building with shear walls and gravity columns using fiber-discretized cross sections in the plastic hinge zone between the first storey and the fourth storey using OpenSees in 3D. The researchers considered a probabilistic model for seismic hazard in Vancouver to perform loss assessment of a Vancouver high rise building. Applying reliability analysis approach, loss curve for the building was derived by combining the results of the behaviour of the building under three different types of ground motions (crustal, sub-crustal and subduction zone).

Wallace (2012) studied behaviour of structural walls and coupling beams to address unexpected observed damage of code conforming shear wall buildings after recent earthquakes. The author reassessed ACI 318 (ACI 2005) provisions for thin walls and suggested an increase in the design displacement equation for shear walls in ACI. Also, the researcher suggested that the walls need to be tension-controlled and detailed as such, for the distribution of plasticity to be consistent with the ACI code. These issues can be addressed by more restrictive limits for wall thickness and slenderness.

Zareian, and Krawinkler (2010) studied the collapse capacity of moment-resisting frame and shear wall structures, quantifying the effects of fundamental period, base shear strength and deformation, while also considering strength deterioration properties of structural components and applying Incremental Dynamic Analysis in Drain 2D. Collapse capacity of the structure (referring to collapse as loss of lateral load resisting capability of the system) was defined as spectral acceleration at fundamental period ( $S_a$ ) when the building experiences dynamic lateral instability. Instead of using a scalar value of intensity measure (such as  $S_a$ ), a vectored-value of  $(S_a, \epsilon)$  was used, where  $\epsilon$  stands for the difference between  $S_a$  of a specific ground motion and the median of the spectral acceleration predicted by an attenuation relationship at the fundamental period of the structure. As expected, structures with longer periods or lower base shear strength had smaller collapse capacity.

Perform 3D, the seismic analysis software used in this study, was applied by Ghodsi and Flores Ruiz (2010) to design a 42 storey shear wall building located in Los Angeles. The researchers used a fiber model for the shear wall. The first design of the building was based on a modal response spectrum analysis and the second design was based on the Los Angeles Tall Building Design Council's prescribed methodology, "An Alternative Design Approach for Tall buildings." This methodology requires the building to satisfy service level criteria - using a modal response spectrum analysis - and collapse prevention criteria using 7 time-history non-linear analyses.

Belletti *et al.* (2013) studied seismic performance of regular multi-storey precast reinforced concrete structural wall buildings having different cross section shapes (rectangular, "U", "L" and "C") considering different modeling approaches for pushover analyses with different levels of accuracy, including shell model (PARC), distributed plasticity model (fiber-element in SeismoStruct) and lumped-plasticity model. The seismic response from the lumped plasticity model was consistent with the results obtained from the other two refined models while PARC model was able to take into account the real crack pattern development.

Another study using Perform 3D software was conducted by Pejovic and Jankovic (2015) to derive seismic fragility of reinforced concrete high rise-buildings (20-storey, 30-storey and 40-storey), representative of high-rise buildings in the Southern euro-Mediterranean zone with core wall structural system. The analysis was conducted for 60 ground motions with a wide range of magnitudes, distances to source and different site conditions. Inter-Storey drift was selected as a damage representative parameter, and threshold of each limit state was defined. It was observed that the fragility of RC high-rise buildings for ground motions having the same PGA but larger magnitudes was higher than the fragility under smaller magnitude earthquakes at each damage state. The authors also concluded that the fragilities of RC high-rise buildings for smaller and larger

distance to source had no significant difference. The analytical fragility curves derived were compatible with expert HAZUS curves.

## **2.2. Previous Studies on Seismic Performance of Different Types of Reinforced Concrete Buildings**

### **2.2.1. Seismic Performance of Code Conforming and Pre-code Reinforced Concrete Buildings with Regular Structural Layouts**

There has been limited research on seismic risk assessment of buildings in Canada. Ventura *et al.* (2005) conducted research on seismic risk estimation in southwestern British Columbia, classifying buildings in British Columbia into 31 categories based on their material, lateral load bearing system, height, use, and age deriving damage probability matrix for each building class. The authors provided fragility curves for each category of the buildings in terms of Modified Mercalli Intensity (MMI). The results showed that unreinforced masonry was the most vulnerable building type, however many of these types of buildings in British Columbia were retrofitted.

Ramamoorthy *et al.* (2006) modelled a two-dimensional frame with nonlinear member behaviour, incorporating degrading hysteretic characteristics. The results were used to derive fragility curves of a two-storey reinforced concrete frame building designed for gravity loads, and another frame with retrofitted (strengthened) columns. The modelling was based on lower bound data. Spectral acceleration at the fundamental period of the structure as hazard intensity parameter was chosen using synthetic ground motions developed for the Memphis region (mid-America) and bilinear demand modelling was assumed. It was shown that the seismic performance of the gravity designed building could be significantly improved by the simple retrofit strategy of column strengthening.

Lupoi *et al.* (2006) presented seismic fragility function of structural systems based on a preliminary, limited simulation applying nonlinear dynamic analyses to assess probabilistic characterization of the demands on the structure. The researchers also compared the results with fragility obtained by Monte Carlo simulation. The proposed model was intended to decrease computational effort. It was applied to a steel-concrete box girder viaduct with RC piers, and a three-dimensional RC building structure subjected to bidirectional excitations.

Ramamoorthy *et al.* (2008) developed fragility curves for reinforced concrete frame buildings designed for gravity loads with various heights (one, two, three, six, and ten-stories). They selected peak inter-storey drift as the response parameter. Capacity values were selected based on performance levels or damage state as specified in FEMA 356 nonlinear pushover analyses. Fragility estimation was formulized as a function of spectral acceleration and the fundamental building period for gravity designed RC buildings with one to ten storey heights.

Celik and Ellingwood (2009) assessed seismic vulnerability of gravity load designed RC frames in Memphis, Tenn. with 2D modelling using OpenSees with synthetic earthquake ground motions. Seismic fragilities were derived for low, mid, and high-rise gravity load designed RC frames and were used to assess the seismic vulnerability of the RC frame inventory in Memphis, based on the performance-based design objectives in FEMA 450. The results indicated that the majority of existing gravity load designed RC were vulnerable based on life safety and collapse prevention performance objectives.

Buratti *et al.* (2010) derived seismic fragility curves for a three storey reinforced concrete frame structure considering uncertainties in structural parameters and seismic excitation, using Response Surface (RS) models with random block effects. They calibrated RS models with numerical data obtained by non-linear incremental dynamic

analyses. Different combinations of RS models and simulation plans were adopted to reach acceptable compromise between the accuracy of the results and the computational effort. A minimum number of 9 different ground-motion time-histories is suggested to account for ground-motion variability with sufficient accuracy. Also, a comparison was made between linear and quadratic RS models.

Mitropoulou and Papadrakakis (2011) applied Neural Network to predict results of Incremental dynamic analysis and conduct fragility assessment of 3D buildings in order to reduce computational effort required for fragility analysis of structural systems. They proposed a soft computing based framework for the fragility assessment of 3D buildings to reduce the order of computational effort required to perform a full fragility analysis.

Masi and Vona (2012) evaluated seismic capacity of existing RC buildings designed for gravity loads only. They used non-linear dynamic analysis of selected structural models based on recent and old building designs, using IDRAC. They considered different building ages, number of storeys, the presence and position of infill walls, and plan dimensions. Seismic response was analyzed by taking into consideration different peak and integral intensity measures, and different response parameters. The results showed that the best intensity measure to be used was the Housner Intensity IH, and that all the response parameters had correlation coefficient values statistically significant with IH, but the best correlation was observed between IH and drift.

Rajeev and Tesfamariam (2012a) considered the effect of soil structure interaction and uncertainty in soil properties on seismic fragilities of non-ductile reinforced concrete frames on dense silty sand. Three, five, and nine-storey three-bay moment resisting reinforced concrete frames were analyzed in 2D using OpenSees and three sets of 10 ground motions with mean spectrum of 100, 500, and 1000 year earthquake return periods. The researchers derived fragility curves for fixed base model and models with soil-structure interaction. Sample of soil properties and foundation parameters were

assumed and the fragility curves were developed. The results showed that soil structure interaction effects had a significant influence on fragilities compared to the fixed base model. However, including uncertainties in soil parameters resulted in a slight change in fragilities. Therefore, assuming median (or mean) values for soil and foundation parameters were found to be sufficient for the purpose of seismic loss estimation.

Bilgin (2013) studied seismic fragility of reinforced concrete public buildings with representative designs conforming to the 1975 version of the Turkish seismic design code. The buildings were designed by nonlinear static analyses in two principal directions of equivalent single degree of freedom systems. Seismic deformation demands were computed from a set of 100 strong ground motion records with Peak ground velocity as seismic intensity measure. The results revealed that the effect of concrete and steel detailing quality was limited on the Immediate Occupancy performance level, especially under increasing load, while this effect was more critical for Life Safety and Collapse Prevention performance levels.

Jeon *et al.* (2015) studied fragility functions for non-ductile concrete frames using numerical simulation focusing on new models for columns and beam-column joints incorporated in OpenSees. The column model included shear-strength under cyclic response. The beam-column joint model included joint strength and cyclic response parameters. Different numerical models were used for a prototype non-ductile concrete frame, including those that accounted for rigid beam-column joints, nonlinear joint shear response, nonlinear joint shear and bond-slip response, and column shear failure. The models were subjected to a set of ground motions representative of the seismic hazard of California to develop fragility curves. A number of conclusions were drawn based on the results of fragility analysis. For the shear response models, median intensity measures and median storey drifts were not significantly affected by component response models. The frame that included shear failure modelling in columns but not in joints showed the highest vulnerability. On the other hand, the frame with rigid joints had the lowest

vulnerability. The models without joint flexibility caused larger seismic demands (likely due to the development of a soft-storey) while seismic demands were distributed over all stories for the models with joint flexibility. Including joint anchorage made a slight increase in seismic demand and the combined joint and column shear response model resulted in similar seismic demands compared to the model with joint shear response model. The authors recommend the simulation of both joint and column shear failures in fragility assessment of non-ductile RC frames.

Pitilakis *et al.* (2014) assessed seismic vulnerability of reinforced concrete buildings having fixed base conditions while incorporating soil-structure interaction. The effects of building age and soil-structure interaction were investigated while accounting for the age of buildings with chloride-induced corrosion (resulting in loss of reinforcement cross-sectional area, degradation of concrete cover and reduction of steel ultimate deformation). Numerical building models were subjected to two dimensional incremental dynamic analyses. Seismic performance of recently built ( $t = 0$  years) and corroded ( $t = 50$  years) RC moment resisting frame structures, designed according to three different seismic code levels, were assessed. The results indicated that overall seismic vulnerability increased over time due to aging effects (corrosion). Furthermore, soil structure interaction effects also resulted in higher vulnerability values.

Karapetrou *et al.* (2015) also studied the effect soil-structure interaction and site soil conditions on seismic performance of reinforced concrete moment resisting frame buildings. A 9-storey RC moment resisting frame, designed for low seismic effects, was chosen as the reference structure. Applying incremental dynamic analyses, fragility curves were derived as a function of peak ground acceleration for both fixed-base and soil structure interactive models (for linear elastic and nonlinear soil behaviour). The results demonstrated that with linear elastic soil behaviour there was little difference between coupled and uncoupled soil structure interaction approach. Nonlinear soil structure interaction resulted in an increase in structural vulnerability compared to the

corresponding fixed-base model. Among the analyzed cases, the uncoupled fixed based model with linear site effects showed the highest vulnerability. This case was identified as the conservative approach with less computational cost.

### **2.2.2. Seismic Performance of Reinforced Concrete Buildings with Irregularities**

Jeong and Elnashai (2007) developed an analytical methodology for derivation of fragility functions for three-dimensional (3D) structures with plan irregularities, employing Park and Ang spatial (3D) damage index. The researchers indicated that the absence of a single quantity to measure the three-dimensional response taking into account torsion and bi-directional responses reduces the reliability of fragility assessment of structures with planar irregularity. The authors suggested a new spatial damage measure in fragility analyses of structures with significant torsional and bi-directional effects. The comparisons between the fragility curves derived by the new spatial damage index and conventional damage indices used in the published literature indicated less conservative estimation of the response by the conventional damage indices.

Athanassiadou (2008) studied multi storey reinforced concrete (R/C) frame buildings with irregularity in elevation. Two ten-storey two-dimensional plane frames with two and four large setbacks in the upper floors and another building which was regular in elevation, all designed based on the provisions of the 2004 Eurocode8 (Eurocode 2005) for high ductility (DCH) and medium ductility (DCM) were considered. The same peak ground acceleration (PGA) and material characteristics were used for both cases. SAP2000 generated models were subjected to inelastic static pushover analysis and inelastic dynamic time-history analysis for assessment of seismic performance based on both global and local criteria. The authors presented the results and showed the effects of vertical irregularities.

Sarkar *et al.* (2010) quantified irregularity, accounting for dynamic characteristics (mass and stiffness). They proposed “regularity index” for assessing the degree of irregularities in a stepped building frame and proposed a modification to code specified empirical formula for estimating fundamental period for regular frames, to estimate the fundamental time period of the stepped building frame as a function of their regularity index.

Rajeev and Tesfamariam (2012b) assessed fragility of structures with three, five, and nine-storey RC frames designed prior to 1970s giving consideration to soft storey and quality of construction (by modelling non-code conforming to RC buildings, mainly designed for gravity loads). The authors used response surface method to develop a predictive equation for PSDM parameters as a function of soft storey and construction quality. They concluded that the results were sensitive to the interaction of soft storey and construction quality. The authors then suggested considering the effects of soft storey (vertical irregularities) and construction quality in the decision for seismic risk assessment.

Jeong *et al.* (2012) conducted fragility analyses of twelve Eurocode-compliant multi-storey reinforced concrete buildings with regular and irregular structural layouts. The buildings had different heights, ductility levels, and design peak ground accelerations. The authors applied Incremental dynamic analyses using ZEUS software in 2D. They concluded that based on the observation of damage state probabilities of wall-frame structures designed to high PGA and ductility levels, this type of buildings did not satisfactorily achieve the favorable safety objectives. Other types of structures satisfied the Life Safety performance level at design intensity, implying that RC frame buildings designed to modern seismic codes are expected to have satisfactory seismic performance. Moment frame buildings with elevation irregularities, designed by according to a modern seismic code, also showed acceptable seismic performance, as these codes have requirements to overcome unfavorable irregularity conditions. The

authors observed no clear trend comparing the limit state probabilities of buildings designed to different ductility levels. Wall-frame buildings designed to higher PGA had relatively low probabilities of minor and moderate damage due to the high stiffness of the shear walls but higher probabilities of severe damage states. In these buildings adopting higher force reduction factors, which is justified by the additional ductility of flexural members, may increase seismic vulnerability. The margin of safety increased with reduction in the design ground acceleration as gravity loads started governing the design. The authors derived a relationship for quantifying the limit state probabilities of medium rise RC buildings designed to modern seismic codes by using the correlation of results obtained from their research.

Varadharajan *et al.* (2013) studied the behaviour of RC moment resisting frames with setbacks, proposing ‘irregularity index’ based on the dynamic characteristics of the frames to quantify the setback irregularity. They modeled 2D Frames with different arrangements of setbacks designed in accordance with the current European standard code, Eurocode 8. They analyzed the buildings using ETABS and 13 ground motions scaled to different intensities to obtain different performance levels as prescribed by SEAOC (SEAOC 1995). They observed significant influence of beam–column strength ratio, number of stories, and geometric irregularity on inelastic seismic demands. Maximum deformations occurred in the vicinity of setbacks and at tower portions of the setbacks.

Gokdemir *et al.* (2013) assessed the effects of torsional irregularity on buildings with different number of storeys, using SAP2000, and concluded that separating big building sections from each other with proper separation distances and increasing lateral rigidity on the weak direction of the structures reduced the effect of this irregularity.

Beigi *et al.* (2015) studied factors affecting repair costs after strong earthquakes for reinforced concrete frame buildings with non-ductile detailing, representative of

construction practices in Italy (expandable to other parts of the world) during the 1950s and 1960s. A six-storey reinforced concrete frame building was numerically simulated for two scenarios consisting of partial or full masonry infills, with soft-storey response developing at the first storey for the partial infill variant. Nonlinear incremental dynamic analyses were conducted and a loss estimation procedure was used to investigate the performance of the systems and the influence of P-Delta effects on peak seismic demands. The results indicated that the partial infill RC structures, in which soft-storey mechanisms form in the open storey, are more prone to collapse. Applying a loss estimation study considering different scenarios, the authors observed that lower losses due to the reduced damage of non-structural components in upper levels resulted in less loss for buildings with soft-storey provided that collapse was prevented at the first storey level. Moreover, losses in soft-storey buildings were mostly of equal value to the repair cost of structural and non-structural components at the soft-storey level. Therefore, an effective retrofit strategy was suggested as increasing the deformation capacity at the soft-storey level without changing the isolating effect that the soft-storey mechanism provides to the levels above. This would also eliminate further interventions in the stories above.

### **2.3. Evolution of Canadian Seismic Codes and Standards**

In order to assess the behaviour of existing buildings in Canada, a review of the evolution of two sets of documents is necessary; i) the evolution of the National Building Code of Canada addressed by Mitchell *et al.* (2010) which gives the level of design seismic loads, i.e. structural demand, and ii) the evolution of seismic design and detailing provisions of the CSA A23.3 Standard addressed by Al-Sadoon (2016) which provides strength and ductility through proportioning and detailing of the members, i.e., capacity.

Assessing the performance of existing RC buildings is much more challenging compared to designing of new buildings, as it requires assessing building behaviour with

limited information on as built conditions, including characteristics of materials and possible deterioration with time.

The design philosophy has changed from working stress design to ultimate strength design over the years, with load factors and capacity reduction factors introduced to allow for safety. The ultimate state design has evolved into limit states design, with the implementation of load factors and material resistance factors to maintain safety. Construction practice has changed significantly, as well. When as built information is not available or complete, the structural codes in force at the time of construction can be the source of information regarding design methods and practices adopted, which permits the categorization of buildings based on their year of design and construction.

### 2.3.1. Evolution of National Building Code of Canada

The seismic design practice in Canada has evolved over the years. The first seismic design provisions were included in the Appendix of the 1941 edition of the National Building Code of Canada (NRCC 1941). The provision was essentially based on the 1935 Uniform Building Code (UBC 1935) of the US, consisting of a lateral design force  $V$ , located at the center of gravity of the building, that varied between 2% and 5% of building weight depending on the bearing capacity of the soil. In the 1953 NBCC, the first seismic zoning map was introduced. The country was divided into four zones of seismic intensity based on the locations of large historical earthquakes with the highest intensity values in the western part of British Columbia, and in the St. Lawrence and Ottawa River valleys. The lateral seismic design force in the 1953 NBCC was obtained by  $F_i = C_i W_i$ ; where  $F_i$  is the storey shear due to the seismic force at the  $i^{\text{th}}$  level,  $W_i$  is the total weight (dead load plus 25% of the design snow load) tributary to the  $i^{\text{th}}$  level, and  $C_i$  is the seismic force coefficient, its minimum value being  $0.15/(N+4.5)$  for Zone 1, where  $N$  is the number of storeys above the  $i^{\text{th}}$  level. The seismic force coefficient for minimum earthquake loads,  $C_i$  is multiplied by 2 for zone 2 and multiplied by 4 for

zone 3. The same equation was provided in the 1960 NBCC (NRCC 1960), with added requirement for consideration of torsional effects, but without any specific guidance provided for consideration of these effects.

The same seismic zone map was used in the 1965 NBCC (NRCC 1965) was the same as the earlier version with the introduction of importance factor, foundation factor; and consideration of torsion that differentiated this version of NBCC from the US codes of the time. The lateral seismic load was calculated based on  $V = RCIFSW$ ; where  $R$  is the seismic regionalization factor with values of 0, 1, 2, and 4 for seismic intensity zones 0, 1, 2, and 3, respectively;  $C$  is the type of construction factor with values of 0.75 for moment resisting frames and reinforced concrete shear walls having adequate reinforcement to behave in ductile manner, and 1.25 for other types of buildings;  $I$  is the importance factor with values of 1.0 and 1.3 (buildings with large assemblies of people, hospitals, and power stations);  $F$  is the foundation factor with values of 1.5 for highly compressible soils and 1.0 for other soil conditions;  $S$  is the structural flexibility factor of  $0.25/(N + 9)$ , where  $N$  is the number of storeys;  $W$  is the total weight (dead load plus 25% snow load plus full live load for storage areas). The total lateral seismic force was assumed to be linearly distributed proportional to the height and weight of the floors. This edition contained torsional design provisions considering torsional eccentricity equal to  $e_x = 1.5e + 0.05D$ ; where  $e$  is the distance between the center of mass and the center of rigidity, and  $D$  is the plan dimension in the direction of the computed eccentricity. In the 1965 NBCC, if  $e_x$  exceeded  $D/4$ , either dynamic analysis was required or the computed torsional moment had to be doubled. The ultimate load combination was calculated by  $U = 1.35(D + L + E)$ , where  $D$ ,  $L$ , and  $E$  are the effects from dead, live, and earthquake loads, respectively.

In the 1970 NBCC (NRCC 1970) a probabilistic seismic zoning map with four zones was used for the first time in Canada based on expected accelerations,  $A_{100}$ ,

having a probability of exceedance of 0.01 (100-year return period). In this edition of the code Montreal and Ottawa changed from zone 3 to zone 2. A structural flexibility factor,  $C$  was introduced depending on the structural period. Higher mode effects were accounted in this edition of NBCC. The minimum lateral seismic force (base shear),  $V$ , was given as  $V = (1/4)RKCIW$ ; where  $R$  is the seismic regionalization factor,  $K$  is the type of construction factor,  $C$  is the structural flexibility factor. For one and two storey buildings  $C=0.1$ , and for other buildings  $C = 0.05T^{1/3}$ ; where  $T$  is the period of the structure. Structural period is calculated as  $T=0.1N$  for moment frames, and  $T = 0.05h_n / D^{1/2}$ ; where  $h_n$  is the height of structure;  $D$  is the dimension of the building parallel to the seismic force; and  $N$  is the number of storeys. This edition was the first Canadian code where the structural response factor was assumed as a function of the structural period.

While the 1970 NBCC referred to ductile moment resisting frames, design and detailing provisions for ductile frame members and ductile flexural walls were not provided until the “Special provisions for seismic design” were introduced in the 1973 CSA A23.3 Standard (CSA 1973). The ultimate load,  $U$ , for seismic design was calculated in the 1970 NBCC based on Equations 2.1 and 2.2.

$$\begin{aligned}
 U &= 1.15D + 1.35(L + E) \\
 U &= 1.5D + 1.8E \\
 U &= 0.9D + 1.35E
 \end{aligned}
 \tag{Eq. 2.1}$$

The same seismic zoning map was used in the 1975 NBCC (NRCC 1975) in which the minimum seismic base shear,  $V$ , was specified as:

$$V = ASKIFW
 \tag{Eq. 2.2}$$

where  $A$  is the horizontal design ground acceleration with values of 0, 0.02, 0.04, and 0.08 for seismic zones 0, 1, 2, and 3, respectively; and  $S$  is the seismic response factor calculated by  $S = 0.5/T^{1/3} \leq 0.1$ .

The torsional eccentricity,  $e_x$  was calculated as  $e_x = 1.5e + 0.05D$  and  $e_x = 0.5e - 0.05D$ . Dynamic analysis was required by the code when  $e_x$  exceeded  $D/4$ ; otherwise the computed torsional moment was to be doubled. The factored load combinations were defined as Equation 2.3:

$$\begin{aligned} U &= 1.25D + 0.7(1.5L + 1.5E) \\ U &= 1.25D + 1.5E \\ U &= 0.85D + 1.5E \end{aligned} \tag{Eq. 2.3}$$

The 1975 NBCC (NRCC 1975) remained essentially the same as the 1970 NBCC except for the introduction of minimum base shear equal to 90% of the static base shear when dynamic analysis was considered in design. The 1980 NBCC (NRCC 1980) used the same seismic zoning map and base shear equation in the 1975 NBCC with modifications for  $S$  values.

New seismic zoning maps, based on the point source model developed by Cornell (Cornell 1968), were introduced in the 1985 NBCC (NRCC 1985). The seismic zoning map was based on a probability of exceedance of 10% in 50 years or 0.0021 per year (return period of 475 years) for seven seismic zones. Seismic base shear was calculated based on Equation 2.4.

$$V = vSKIFW \tag{Eq. 2.4}$$

where  $v$  is the velocity zonal ratio and  $S$  is the seismic response factor, which had an improved equation. The period of the structure was determined as  $T = 0.09h_n / D^{1/2}$  with the exception of moment-resisting space frames, for which  $T = 0.1N$ . The use of period from modal analysis was allowed for the first time in the 1985 NBCC, but was limited to 1.2 times the static period. Dynamic base shear was limited to a minimum base shear value of 90% the static value. The torsional design eccentricity changed as in Equation 2.5, while doubling the torsional moment was no longer required:

$$\begin{aligned} e_x &= 1.5e + 0.1D \\ e_x &= 0.5e - 0.1D \end{aligned} \qquad \text{Eq. 2.5}$$

The 1990 and 1995 NBCC (NRCC 1990, 1995) used the same seismic zoning map as in 1985NBCC. Significant changes were however introduced, and included the replacement of the  $K$  factor by the force modification factor  $R$ . The base shear was determined by  $V = U(vSIFW) / R$ ; where  $U$  was a calibration factor ( $U = 0.6$ ) in order to insure consistency with the previous codes,  $S$  was the seismic response factor with modified definitive equations based on the structural period.  $I$  was the importance factor and  $F$  was the soil modification factor. The  $R$  factor reflected the capability of a structure to dissipate energy through inelastic behaviour. The  $R$  factor varied between 1.0 for unreinforced masonry and 4.0 for ductile moment-resisting space frames. Intermediate values of  $R$  were introduced with  $R = 2.0$  for nominally ductile walls, concrete frames, and braced steel frames.

The earthquake load factor was reduced to 1.0 to reflect the extreme character of the loading considered in design, resulting in the load combinations for earthquake design represented in Equation 2.6.

$$\begin{aligned}
 U &= 1.25D + 0.7(1.5L + 1.0E) \\
 U &= 1.25D + 1.0E \\
 U &= 0.85D + 1.0E
 \end{aligned}
 \tag{Eq. 2.6}$$

The inter-storey drift ratio obtained from analysis and multiplied by R factor was to be limited to 0.01 for post disaster buildings and 0.02 for other types of buildings.

The 1995 NBCC had three major changes compared to the 1990 version. These included the introduction of new R factors, new equations for calculating structural period, and new equations for considering torsional eccentricity. The load combination format also changed to Equation 2.7.

$$\begin{aligned}
 U &= 1.0D + 1.0L \\
 U &= 1.0D + 1.0E + 1.0L \\
 U &= 1.0D + 1.0E + 0.5L
 \end{aligned}
 \tag{Eq. 2.7}$$

The 2005 and 2010 NBCC (NRCC 2005, 2010) had several major changes compared to the earlier editions of the code. The uniform hazard spectrum (UHS) approach (Cornell 1968) was adopted essentially giving site-specific response spectral accelerations for numerous locations in Canada (Adams and Atkinson 2003). These spectral accelerations had a probability of exceedance of 2% in 50 years (2475-year return period). This lower probability provided a more uniform margin of collapse. The dynamic analysis approach became the preferred method of analysis and had to be used for structures with certain types of irregularities. The minimum lateral earthquake design force,  $V$ , at the base of the structure (equivalent static force procedure), was to be obtained from Equation 2.8.

$$V = S(T_d)M_v I_E W / R_d R_0
 \tag{Eq. 2.8}$$

where  $M_v$  accounts for the higher mode effect and  $I_E$  is the importance factor. The structural period of a shear wall building was to be calculated by the code formula or using dynamic analysis which had limitation related to the type of seismic force resisting system (SFRS).

The 2005 and 2010 NBCC allow using linear dynamic analysis by either the Modal Response Spectrum Method or the Numerical Integration Linear time history Method and Nonlinear Dynamic Analysis. The ground motion histories used in Time History Method must be compatible with the response spectrum constructed from design spectral acceleration values. The design elastic based shear,  $V_{ed}$  shall be multiplied by  $I_E$  and divided by  $R_d$  and  $R_o$  to get the design base shear  $V_d$  for regular structures. If  $V_d$  from the dynamic analysis is less than 80% of the static  $V$ ; 80% of the static  $V$  shall then be used instead. (For irregular structure maximum of  $V_d$  and  $V$  shall be used).

For structures with a fundamental period higher than 0.7 s (which is applicable to the 10-storey shear wall building in this study), a portion of the shear force equal to  $F_t = 0.07T_d V$  and limited to  $0.25V$  is applied as a concentrated load at the top.  $F_t$  is assumed to be 0 for buildings for lower period. The lateral load at each storey level is calculated as Equation 2.9.

$$F_x = (V - F_t)W_x h_x / \left( \sum_{i=1}^n W_i h_i \right) \quad \text{Eq. 2.9}$$

where  $h_i$  and  $W_i$  account for storey heights and weights respectively.

Lateral deflections obtained from a linear inelastic analysis while incorporating the effect of torsion must be multiplied by  $R_d R_o / I_E$ . The inter-storey drift ratio must be

limited to 0.01 for post-disaster buildings, 0.01 for high importance category buildings and 0.025 for other buildings.

### **2.3.2. Evolution of Canadian Seismic Design and Detailing Requirements for Reinforced Concrete Structures**

CSA Standards for concrete and reinforced concrete were published in 1959 (CSA 1959) as four separate standards (A23.1, A23.2, A23.3 and A23.4 with A23.3 addressing reinforced concrete). They included definitions, general requirements, requirements for concrete materials, reinforced concrete design, and methods of concrete construction, and methods of test and standard practices for concrete standard specification. This edition of the standard was published under the name of “Code of recommended practice for reinforced concrete design,” and conformed to the requirements of the American Concrete Institute’s ACI 318-51. The design requirements were based on allowable concrete and reinforcement stresses (working stress method), and considered both dead and live loads. However, the 1960 NBCC and 1965 NBCC required reinforced concrete structures to conform to the ACI 318-56 (ACI 1956) and ACI 318-63 (ACI 1963), respectively, without making references to the CSA Standards. The CSA Standard A23.3 was published under the name of "Code for the design of plain or reinforced concrete structures" in 1970, and was included in the NBCC Supplement No. 4 of the Canadian Structural Design Manual (NRCC 1970). The ultimate strength design method was included for the first time as an alternative design approach. In addition, capacity reduction factors,  $\Phi = 0.90$  for flexure, 0.85 for diagonal tension, bond and anchorage, and 0.75 for all types of columns were required to be applied to ultimate strengths to compute design capacities. None of the above-mentioned editions of the standard included seismic design provisions.

The first seismic design provision for ductile design was introduced in CSA A23.3-1973 (CSA 1973), which was referenced by the 1975 NBCC (NRCC 1975). The ductility

requirements in NBCC were ensured through seismic design and detailing requirements specified in CSA A23.3-73. As the 1975 NBCC was the first edition of the code that accounted for ductility by introducing the base shear parameter (K) for seven building systems with different ductility characteristics, 1975 is considered as the benchmark code with a significant change in the seismic design practice in Canada.

The “special provisions for seismic design” in CSA A23.3-73 were intended for buildings designed to resist earthquake forces in a ductile manner. The provisions included general requirements and assumptions, special design and detailing requirements for frame elements, connections, and walls. The Standard emphasized on reinforcement detailing for concrete confinement, prevention of longitudinal reinforcement buckling, minimum percentages for flexural reinforcement, longitudinal reinforcement detailing, joint shear, prevention of brittle shear failure and sequence of energy dissipation among the members. The requirements were essentially the same as those in ACI 318-71 (ACI 1971) building code in the US.

Regarding shear walls, requirements of ductile flexural walls were explained in more details in the Commentary of CSA Standard A23.3-73 relative to the requirements of the ACI 318 Building Code. A design procedure was given in Appendix A of the commentary to insure sufficient ductility in a flexural wall. The wall ductility detailing provisions covered a number of topics, including: minimum amount of distributed vertical and horizontal reinforcement; concentrating an adequate amount of flexural vertical reinforcement at both wall ends that will provide moment capacity in the wall, and the possibility of plastic hinge formation; reinforcement splices at wall ends to prevent bond failure of tension reinforcement; and bar buckling ties at wall ends to ensure yielding of compression reinforcement without buckling. To ensure adequate protection against local break-down during the formation of plastic hinges, the A23.3-73 required minimum areas of distributed horizontal and vertical reinforcement (0.0025 and 0.0015 times the cross sectional area of the wall, respectively). Also, the maximum

allowable spacing of distributed steel was set to 18 inches (450 mm) except for the lower half of the structure where the value spacing was reduced to 12 inches (300 mm). Minimum flexural reinforcement was defined for both ends of the walls to provide moment capacity required under earthquake force reversals. The standard required each section of the wall to be designed to have shear capacity,  $V_{uc}$ , of 1.1 times the shear force that associated with the moment capacity,  $M_{uc}$ , acting on the wall to prevent shear failure in the plastic hinge region.

CSA A23.3-77 (CSA 1977), referred in the 1980 NBCC was slightly modified compared to its earlier edition. More substantial modifications were introduced to CSA A23.3-84 (CSA 1984), which was referenced in the 1985 NBCC. Some requirements superseded the earlier ones for spacing of transverse reinforcement. Hoop reinforcement was specified for plastic hinge regions. The concept of strong column and weak beam energy dissipation criteria was introduced. To achieve this, the total factored resistance of the columns, based on  $\Phi_c=0.6$  and  $\Phi_s= 0.85$ , was required to be at least 10% greater than the total nominal resistance, based on  $\Phi_c=1.0$  and  $\Phi_s= 1.0$ , of the beams framing into the joint.

The provisions of ductile flexural walls in CSA A23.3 M84 were suggested based on comparable provisions in the New Zealand Standard NZS3101 Part 1 (NZS3101 1982), “Code of Practice for the Design of Concrete Structures” (Explanatory Notes on CSA Standard A23.3-M84). The coupled wall requirements were added to the previous ductile flexural wall requirements.

Dimensional stability of ductile walls in potential plastic hinge regions was regulated in the Standard. To avoid excessive congestion within any region of concentrated longitudinal reinforcement, the Standard specified a maximum reinforcement ratio of 6%. Also, the diameter of the bar was set as not to exceed 10% of the wall thickness at the bar location.

Wall horizontal reinforcement were specified to be extended to the ends of the wall at the concentrated reinforcement region and anchored to develop the yielding stress  $f_y$ . At least two curtains were required if the factored shear force exceeded  $(0.2 \lambda \Phi_c \sqrt{f'_c} A_{cv})$  where  $A_{cv}$  is the net area of concrete section, bounded by web thickness and length of section in the direction of the lateral force considered, in ( $\text{mm}^2$ ).

The concentrated reinforcement requirements in regions of plastic hinges were also modified. The spacing between ties for the concentrated reinforcement were required to be less than the least of the 6 longitudinal bar diameters; 24 tie diameters;  $\frac{1}{2}$  the wall thickness,  $b_w$ , or the spacing required for wall ductility.

The wall ductility computation was introduced for the first time in this edition of CSA Standard. The value 0.005 was specified as the minimum reinforcement ratio at any part of this confined region. The clause of “Building Members Requiring Nominal Ductility” was also introduced in this version of CSA A23.3 describing detailing requirements of beams, columns, and walls. The requirements were less stringent compared to the requirements of fully ductile members.

CSA A23.3-94 (CSA 1994) was published under the name of “Design of Concrete Structures” with no major changes compared to the previous Standard. Various design and detailing requirements were added to CSA A23.3-04 (CSA 2004) to cover the newly introduced “Moderately ductile” category. Additional requirements were also introduced for the new seismic force resisting systems (SFRS) incorporated in the NBCC 2005 (NRCC 2005). The Standard included six standard ductile systems and two conventional construction systems. The resistance factor of concrete,  $\Phi_c$ , was increased from 0.6, previously used, to a value of 0.65 with an exception for precast concrete elements produced in manufacturing plants for which a value of 0.7 was specified. The material resistance factor for steel,  $\Phi_s$ , remained unchanged as 0.85.

New ductility provisions were introduced for ductile walls, with/without openings, coupled or partially coupled shear walls, and coupling beams. Short walls with height over width,  $h_w/l_w, \leq 2.0$  were defined as a separate category of “squat walls” as they are more likely to develop plastic shear deformations rather than those due to flexure. These walls had to be designed using  $R_d=2.0$  and  $R_0=1.4$ . The structural irregularity type specified in the code was used for the wall ductility provisions.

The walls were required to be detailed for plastic hinges over a height equal to at least 1.5 times the length of the longest wall above the design critical section. The flexural and shear reinforcement required for the critical section were to be maintained over the plastic hinge length. For all elevations above the plastic hinge region, the design overturning moments was to be increased by the ratio of the factored moment resistance to the factored moment, both calculated at the top of the plastic hinge region. Buildings having stiffness or geometric irregularity were required to be detailed by following comparable uniform building requirements at each irregularity, continuing with 1.5 times the length of the longest wall above and below the irregularities.

Factored shear resistance of walls were required to be greater than the shear force caused by the effects of factored loads, including the effect of magnification of shear due to inelastic effects associated with higher modes. In addition, factored shear resistance was required to be taken higher than the smaller of two the shear corresponding to the development of probable moment capacity of the wall system at its plastic hinge location and the shear resulting from the design load combinations that include earthquake using  $R_d R_o$  equal to 1.0.

In addition, tie requirements for vertical distributed reinforcement was introduced and set as being similar to non-seismic provision for compression members. In addition to 300 mm maximum spacing of distributed reinforcement in regions of plastic hinging as required in the previous Standard, buckling prevention provisions to account for

anticipated reverse cycling yielding was introduced when the area of vertical steel was greater than  $0.005 A_g$  and the max bar size used is greater than 15M. When factored shear force at hinging region of the wall exceeded  $(0.18 \lambda \Phi_c \sqrt{f'_c} A_{cv})$ , the Standard required using at least two curtains of reinforcement. The steel yield stress that needed to be developed in regions of plastic hinging was increased from  $f_y$  to  $1.25 f_y$ . The minimum area of concentrated vertical reinforcement to ensure post cracking capacity in plastic hinge regions was reduced from  $0.002 b_w l_w$  in previous Standard to  $0.0015 b_w l_w$  for each end of the wall.

The Standard introduced new ductility provisions for ductile individual shear walls, as well as coupled and partially coupled shear walls, superseding the previous requirements. The new provisions required an estimate of the displacement demand of the seismic force resisting system under the design earthquake and was less restrictive than the old provisions for ductile walls while it was considerably more restrictive for moderately (nominally) ductile walls (Adebar *et al.* 2005). The concept of limit state was introduced by setting the inelastic rotational capacities of both walls and coupling beams to be greater than their respective demands. The plastic hinge length was set to  $0.5l_w$  for the rotational capacity and  $l_w$  for rotational demand. 0.025 was set as the upper limit of inelastic rotation capacity governed by tension steel strain while 0.004 limit value was assigned as the minimum inelastic rotational demand to insure minimum level of ductility in the building.

Similar but less stringent requirements were set for moderately ductile shear walls compared to those for fully ductile walls. Detailing requirements of conventional construction structures, of  $R_d = 1.5$ , were also introduced in the CSA A23.3 2004. Frame members that are not considered part of the seismic force resisting system were required to be checked for compatibility of displacements to ensure that these parts will continue

to function at anticipated displacement levels during earthquake (Explanatory Notes on CSA Standard A23.3-04).

#### **2.4. Seismic Evaluation and Retrofit of Existing Buildings using ASCE41**

The American Society of Civil Engineers provides a comprehensive guideline on seismic evaluation and retrofitting of buildings in ASCE 41 (ASCE 2007). This document suggests procedures that go beyond those outlined in current codes while assessing the behaviour of existing buildings. Performance objectives of building codes, on the other hand, are intended for designing new buildings with requirements on building configuration, strength, stiffness, detailing, special inspection and testing. Although these strength and stiffness requirements can be applied to existing buildings, the other provisions are not easily adopted for existing buildings. This primarily because the seismic-force-resisting elements of an existing building do not have construction details similar to the new construction requirements, and the basic assumptions resulting in ductile behaviour is not met in existing buildings.

ASCE 41 identifies potential seismic deficiencies in existing buildings giving the owner the option of choosing acceptable risk unless the evaluation is enforced by a local ordinance for a hazard-reduction program or is made mandatory by a regulation, building code, or policy, forcing the owner to choose whether to retrofit or demolish the building, or to place a limit on the occupancy. In order to perform seismic evaluation and retrofitting, it is necessary to first define the intended performance objectives. A Performance Objective consists of selected pairs of Seismic Hazard Level and target Structural and Non-Structural Performance Levels. Performance Objectives could be basic, enhanced, limited, or partial objectives, or an objective equivalent to that in the new building codes.

FEMA 274 (ATC 1997) discusses performance objectives when combining various Performance and Seismic Hazard Levels that need to be cost-effective. This standard presents Performance Objectives, including specific objectives similar to the performance objectives of buildings designed based on new building standards and also specific objectives similar to historically

accepted performance objectives called “reduced code performance” in documents such as the International Existing Building Code (IBC 2012). These performance objectives provide Structural and Non-structural Performance Levels at pre-defined Seismic Hazard Levels.

Building performance can also be qualitatively described based on the safety of the building occupants during and after the seismic event; the cost and feasibility of restoring the building to pre-earthquake state; the down time; and other economic, architectural, or historic criteria. These performance characteristics can be related to the extent of damage that would be sustained by the building in the seismic event.

ASCE 41 represents possible performance objectives that might be considered for a typical building as shown in Table 2-1. Basic Performance Objective for Existing Buildings (BPOE) is a specific performance objective changing with Risk Category, where the Risk Category can be defined by the governing regulations, building code, or policy, or based on ASCE 41 procedures. “a” through “l” refer to different performance objectives to be selected by the user. A combination of performance objectives demonstrated as “a” through “l” may be intended.

**Table 2-1: Performance Objectives (ASCE 2007)**

Seismic Hazard Level	Target Building Performance Levels			
	Operational	Immediate Occupancy	Life Safety	Collapse Prevention
50% in 50 years	a	b	c	d
20% in 50 years	e	f	g	h
Maximum credible return period	i	j	k	l

**Each cell is a discrete Performance Objective**

In summary, ASCE process of seismic evaluation consists of the following parts: Selection of Performance Objective and Level of Seismicity, gathering As-Built Information, description of evaluation Procedures and Evaluation Report. It also gives seismic retrofit process including: Selection of Performance Objective and Level of Seismicity, As-Built Information, Retrofit Procedures, Retrofit Strategies (Local Modification of Components, Removal or Reduction of

Existing Irregularities, Global Structural Stiffening, Global Structural Strengthening, Mass Reduction, Seismic Isolation, Supplemental Energy Dissipation), Retrofit Measures and Verification, Construction Documents and Construction Quality Assurance.

ASCE 41 also has a discussion on types of irregularities including weak storey, soft storey, vertical irregularities, irregularities in geometry and mass, and torsional irregularities. Because it is intended to focus on performance of irregular shear wall buildings in this research, description of the requirement that must be met to classify a building as regular based on ASCE is summarized in this section.

**Weak storey:** The sum of the shear strengths of the seismic-force-resisting system in any storey in each direction is not less than 80% of the strength in the adjacent storey above. The storey strength is the total strength of all the seismic force-resisting elements in a given storey for the direction under consideration (the shear capacity of columns or shear walls or the horizontal component of the capacity of diagonal braces). If the columns are flexure controlled, the shear strength is the shear corresponding to the flexural strength. Weak stories are usually found where vertical discontinuities exist or where member size or reinforcement has been reduced. Weak storey induces in a concentration of inelastic activity which may result in partial or total collapse of the storey.

**Soft storey:** The stiffness of the seismic-force-resisting-system in any storey is not less than 70% of the seismic-force-resisting system stiffness in an adjacent storey above or less than 80% of the average seismic-force-resisting-system stiffness of the three stories above. Soft storey usually occurs in commercial buildings with open fronts at ground-floor storefronts and hotels or office buildings with particularly tall first stories. Soft stories usually result in an abrupt change in storey drift. A tall story or a change in the type of seismic-force-resisting-system is an obvious indication that a soft storey might exist.

A gradual reduction of seismic-force-resisting elements as the building increases in height is typical and is not considered a soft storey condition. The difference between “soft” and “weak” stories is the difference between stiffness and strength. A change in column size can affect strength and stiffness, and both need to be considered.

**Vertical irregularities:** All vertical elements in the seismic-force-resisting-system are continuous to the foundation. The most common example of vertical irregularities is a discontinuous shear wall or braced frame. Vertical discontinuities can be detected by visual observation when the element is not continuous to the foundation, and stops at an upper level. The shear at this level is transferred through the diaphragm to other resisting elements below. The overturning forces that develop in the element continue down through supporting columns. This issue is a local strength and ductility problem below the discontinuous elements, not a global storey strength or stiffness irregularity. The concern is that the wall or braced frame may have more shear capacity than was considered in the design. These capacities impose overturning forces that could overwhelm the columns. Although the strut or connecting diaphragm may be adequate to transfer shear forces to adjacent elements, the columns that support vertical loads are the most critical.

**Geometric irregularities:** Regular buildings do not have changes in horizontal dimension of the seismic-force-resisting system of more than 30% in a storey relative to adjacent stories, excluding one storey penthouses and mezzanines. Geometric irregularities are usually detected in an examination of the storey-to-storey variation in the dimensions of the seismic-force-resisting system. A building with upper stories set back from a broader base structure is a common example. Another example is a storey in a high-rise that is set back for architectural reasons. It should be noted that the irregularity of concern is in the dimensions of the seismic-force-resisting-system, not in the dimensions of the envelope of the building, and, as such, it may not be obvious.

Geometric irregularities affect the dynamic response of the structure and may lead to unexpected higher mode effects and concentrations of demand. A dynamic analysis should be performed for irregular structures to more accurately calculate the distribution of seismic forces. One-storey penthouses need not be considered except for the added mass.

**Mass irregularities:** The effective mass does not change by more than 50% from one storey to the next, then the structure is considered to be regular from the mass distribution perspective. Light roofs, penthouses, and mezzanines need not be considered in this definition. Mass irregularities can be detected by comparison of the storey weights. The effective mass consists of the dead load of the structure tributary to each level, plus the actual weights of partitions and permanent equipment at each floor. Buildings are typically designed for primary mode effects. The validity of this approximation is dependent on the vertical distribution of mass and the stiffness in the building. Mass irregularities affect the dynamic response of the structure and may lead to unexpected higher mode effects and concentrations of demand.

**Torsional irregularities:** If the estimated distance between the storey center of mass and the storey center of rigidity is less than 20% of the building width in either plan dimension, then the building is considered to have negligible torsional effects. Wherever there is significant torsion in a building, the concern is for additional seismic demands and lateral drifts forces.

## 2.5. Incremental Dynamic Analysis

Incremental dynamic analysis (IDA) (Vamvatsikos and Cornell 2002a) is a recently emerged method to estimate structural performance under different levels of seismic loads. It involves subjecting a structural model to a set of ground motion records, each scaled to multiple levels of intensity, producing curves of response parameter versus

intensity level. Advances in computational processing have enabled the application of accurate and complex analysis methods. Therefore, the trend moved from elastic static analysis to dynamic elastic, non-linear static and finally non-linear dynamic analysis. The application of nonlinear analysis results in a single point for design check, while non-linear static pushover gives a ‘continuous’ view of the structural behaviour, from elasticity to yielding and finally collapse. Finally, Incremental dynamic analysis (IDA) is beneficial in both ways. IDA gives thorough understanding of the range of response versus the range of potential ground motion intensity. The structural behaviour under low, moderate and severe ground motions can be assessed, in addition to the estimation of the dynamic capacity of the global structural system.

To illustrate the concept of IDA, a complete application of the methodology was presented by Vamvatsikos and Cornell (Vamvatsikos and Cornell 2002b, 2004) by means of an example 9-storey steel moment-resisting frame. Its application in Performance-Based Earthquake Engineering (PBEE) was also discussed by the authors. An important task in Performance-Based Earthquake Engineering is the estimation of structural behaviour under seismic excitations and finding the mean annual rate of exceeding a predefined level of structural demand (e.g., inter-storey drift ratio) or a certain limit-state capacity (e.g., global dynamic instability). Incremental Dynamic Analysis (IDA) enables such estimations. IDA curves depict the structural response, as measured by a Damage Measure (DM, e.g., peak roof drift or maximum inter-storey drift  $q_{\max}$ ), versus the ground motion intensity level referred as Intensity Measure (IM, e.g., peak ground acceleration or the 5%-damped first-mode spectral acceleration  $S_d(T_1;5\%)$ ). Moreover, these curves can be used to define or justify the definition of limit-states (e.g., Immediate Occupancy or Collapse Prevention). Finally the results can be integrated with the site hazard curve in order to calculate annual rates of exceeding a certain limit-state.

Vamvatsikos and Cornell describe the following steps to apply the methodology:

- *Structural simulation and ground motion records:* Based on previous studies for mid-rise buildings, a set of ten to twenty records are enough to provide sufficient accuracy in the estimation of seismic demands, assuming a relatively efficient intensity measure (IM), like  $S_a(T_1;5\%)$ , is used. The authors selected a set of twenty ground motion records with relatively large magnitudes of 6.5–6.9 and moderate distances, recorded on firm soil.
- *Performing the analysis:* The analyses should be performed with appropriate scaling levels to cover the entire range of structural response, from elasticity, to yielding, and to global dynamic instability. The application of the hunt & fill algorithm is suggested in order to obtain the scaling of earthquake records with a minimum number of runs. Analyses should be first performed at rapidly increasing levels of IM until numerical non-convergence occurs (indicating global dynamic instability), and then additional analyses should be applied for intermediate IM-levels to sufficiently cover the whole range of behaviour.
- *Post processing:* An important task in the process is to choose a suitable pair of quantities in the form of intensity measure and demand measure. For the steel moment-resisting frame studied, the authors chose 5%-damped first-mode spectral acceleration  $S_a(T_1;5\%)$  as IM, which satisfies efficiency (by minimizing the scatter in the results, and requiring only a few ground motion records to provide good demand and capacity estimates) and sufficiency (by providing a complete characterization of the response without the need for magnitude or distance information). DM needs to be application-specific; therefore, peak inter-storey drift ratio was chosen, relating well to global dynamic instability and structural performance limit states.

Based on the results, IDA curves were generated by interpolating between results, allowing the calculation of DM values at arbitrary levels of IM. Finally, the collapse condition is reached when the structure responds with practically “infinite” IM values and numerical non-convergence has been encountered during the analysis, which indicates global dynamic instability (small increments in the IM-level result in unlimited

increase of the DM-response). Three limit-states of Immediate Occupancy, Collapse prevention and global dynamic instability collapse are demonstrated on IDAs. Based on FEMA 350, the Immediate Occupancy limit state of steel moment-resisting frame occurs at a drift of 2%. Collapse Prevention limit state is reached when either the local tangent to the IDA curve reaches 20% of the elastic slope or drift exceeds 10%, whichever occurs first. The global dynamic instability produces a flat-line in the IDA curve and any increase in the IM results in practically infinite DM response. If this point occurs first, it marks the collapse prevention point. Due to record-to-record variability of the results, IDAs were summarized by three curves with 16%, 50% and 84% probability of exceedance. Application of IDA in performance-based-earthquake-engineering framework proposed by FEMA 350 or by the Pacific Earthquake Engineering Center was then discussed, which involves calculating the annual rate of exceeding values of the chosen IM, from Probabilistic Seismic Hazard Analysis, and integrating with the conditional probabilities of exceeding each limit-state to produce the desired annual rates of limit-state exceedance.

## 2.6. Fragility Analysis and Seismic Risk Assessment

The Pacific Earthquake Engineering Center (Cornell and Krawinkler 2000) gives a summarized equation for the probabilistic seismic risk assessment framework.

This framework suggests a relationship between a certain key Decision Variable,  $DV$ , such as annual seismic loss and a series of conditional probabilities, de-aggregating mean annual frequency ( $MAF$ ) of exceeding the decision variable,  $\lambda_{DV}$ , in terms of Damage Measure,  $DM$ , and ground motion Intensity Measure,  $IM$  given in Equation 2.10.

$$\lambda_{DV} = \iint G(DV | DM) dg(DM | IM) d\lambda_{IM} \quad \text{Eq. 2.10}$$

where,  $(DV|DM)$  is the probability that the decision variable exceeds specified values given that the engineering Damage Measures (herein, first storey drifts) are equal to particular values,  $G(DM|IM)$  is the probability that the Damage Measures exceed these values given that the Intensity Measure (herein, spectral acceleration at the fundamental period) is equal particular values, and  $\lambda_{IM}$  is the *MAF* of the ground motion Intensity Measure.

The part  $G(DM|IM)$  in the de-aggregated equation is called the Seismic Fragility function. Seismic fragility relates the probability of reaching or exceeding predefined levels of damage to the severity of ground motion intensity. Analytical fragility curves are derived from the results of numerical simulations of the structure under artificial or historical earthquake records. The fragility function is described in the form of Equation 2.11.

$$P[D > Di | IM] = \phi \left[ \frac{\ln(x / Di)}{\sqrt{\beta_{d/IM}^2 + \beta_c^2 + \beta_m^2}} \right] \quad \text{Eq. 2.11}$$

In which  $\Phi(.)$  is the standard normal cumulative distribution function,  $Di$  is the upper bound for each damage level,  $x$  is the median value of demand as the function of  $IM$ ,  $\beta_{d/IM}$  is the dispersion (logarithmic standard deviation) of the demand conditioned on the  $IM$ ,  $\beta_c$  is the capacity uncertainty and  $\beta_m$  is the modeling uncertainty.

# **Chapter 3. Analytical Modelling of Shear Wall Buildings for Dynamic Analysis**

In this chapter, the focus is placed on modelling shear wall buildings for using with computer software PERFORM-3D for nonlinear time history analysis. The behaviour of shear walls was modeled using fiber-discretized sections for flexural response based on constitutive models for concrete and reinforcing steel. The nonlinear behaviour in shear was modelled on the basis of sectional non-linear shear response specified by the user. The analytical model was validated against experimental results available in the literature. The gravity load frames in the buildings were modelled using beam and column models available in the software, consisting of elastic segments and potential plastic hinges at member ends. Further details of the modelling techniques are discussed in this Chapter.

## **3.1. Shear Wall Element Modelling**

The response of shear wall buildings is primarily dominated by the behaviour of shear walls, which necessitates the development of accurate analytical models for dynamic inelastic analysis. A fiber-discretized model is employed in software PERFORM-3D to simulate the wall behaviour. The modelling of hysteretic behaviour using the fiber model

is considered to be more accurate as compared to the assumption of a single element with concentrated hinges, as it will be able to simulate the progression of plastification within the plastic hinge region more accurately.

The overall behaviour of a wall section in the fiber model is obtained by integrating contributions coming from concrete and steel to sectional resistance. Confined concrete was used in wall boundary elements and unconfined concrete was used for the rest of the shear wall. Confined concrete model developed by Saatcioglu and Razvi (1992) and unconfined concrete model developed by Hognestad (1951) were employed in the sectional analysis. The confined concrete properties were computed based on the spacing and arrangement of boundary element transverse reinforcement. Different levels of confinement were used depending on the detailing of boundary elements when used for moderately ductile and fully ductile shear walls. Figure 3-1 demonstrates the concrete models implemented into the software. Figure 3-2 depicts the input screen of PERFORM-3D for material specification.

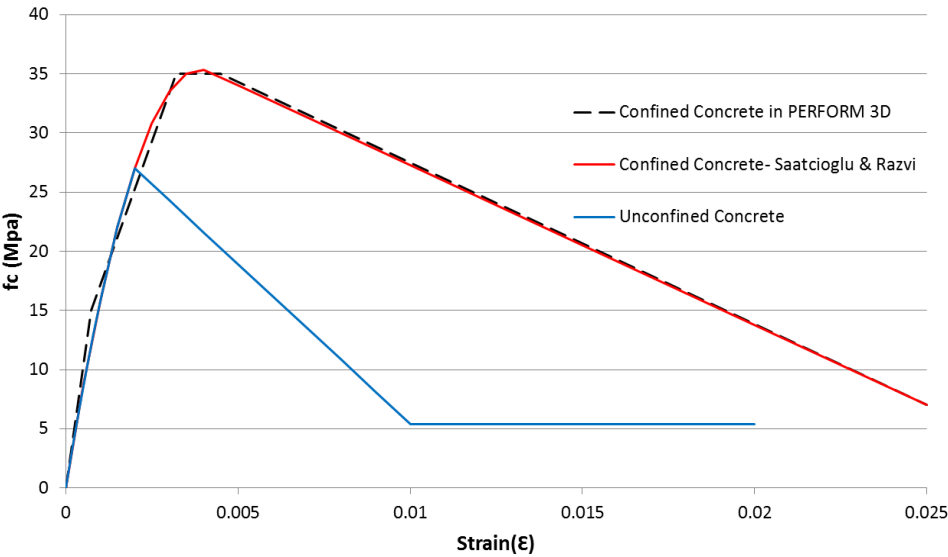


Figure 3-1 Confined and unconfined concrete models used in numerical simulation

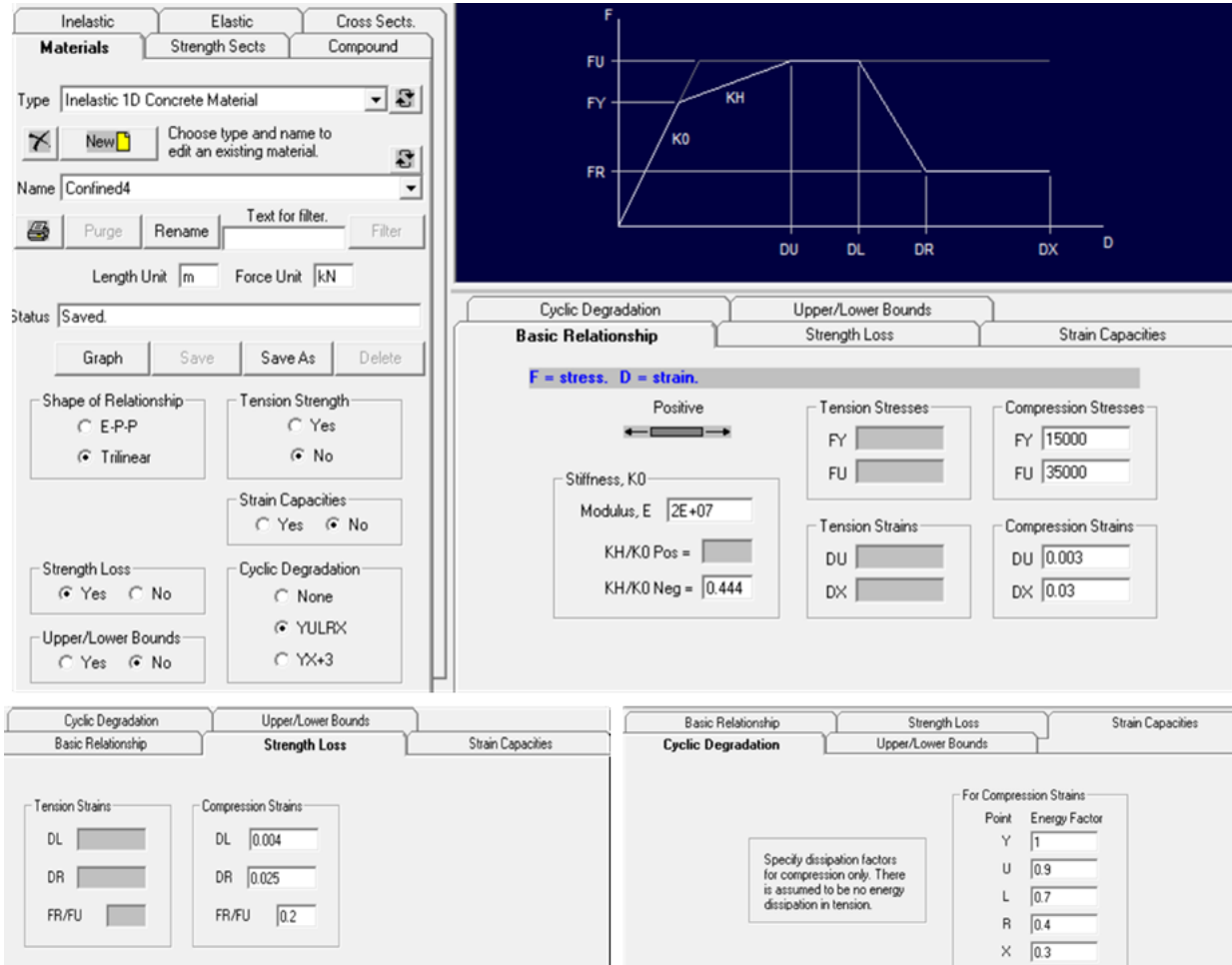


Figure 3-2: Sample concrete material modelling parameters in PERFORM 3D

The stress-strain relationship of reinforcing steel in tension has three linear segments, consisting of elastic, yield plateau, and strain-hardening segments. The behaviour in compression can be different depending on the tie spacing and the buckling characteristics of compression bars. The stress-strain relationship specified by Yalcin and Saatcioglu (2000) was adopted to simulate the behaviour of reinforcement in compression. Accordingly, the reinforcement aspect ratio, defined as the ratio of tie spacing to bar diameter, higher than 8 results in bar buckling. In this case the steel stress drops linearly immediately after yielding with increasing compressive strains. For bar

aspect ratios between 4.5 and 8, the stress-strain relationship develops different levels of strain-hardening, reaching to that specified for tensile strength level in compression when the ratio is 4.5.

Non-linear behaviour of shear is specified in Perform 3D in the form of a shear stress-shear strain relationship. This relationship reflects the combined effects of concrete and steel through a smeared model that consists of linear segments; simulating elastic, post cracking, post-yield and strength decay regions. The initial elastic slope of the relationship is defined as  $G = 0.4E$  up to  $V_c/(b_w d_v)$ , where  $V_c$  is the shear force resisted by the concrete,  $b_w$  is the web width and  $d_v$  is the effective shear depth of the section. The second linear segment has a slope equal to 6% of the initial elastic slope, reflecting concrete cracking up to the combined shear resistance of concrete and shear reinforcement ( $V_c+V_s$ ). These quantities can be computed by following the equations specified in CSA A23.3 (CSA 2014). The third line segment represents the yield plateau region of reinforcement and has zero slope up to the onset of shear strength decay. The strength decay point and the descending branch of the shear stress-strain relationship (decay slope) were estimated from observations made on shear wall tests conducted at the Portland Cement Association (Oesterle 1976). Accordingly, the walls showed shear strength decay starting at a shear strain value of about 0.01 to 0.02, depending on the level of detailing used. In the current models, the strength decay point was assumed to start at 0.0150 and 0.0175 for moderately ductile and fully ductile walls. Figure 3-3 shows the comparison of shear force-shear strain envelop curves for experimentally recorded and analytically generated hysteretic relationships.

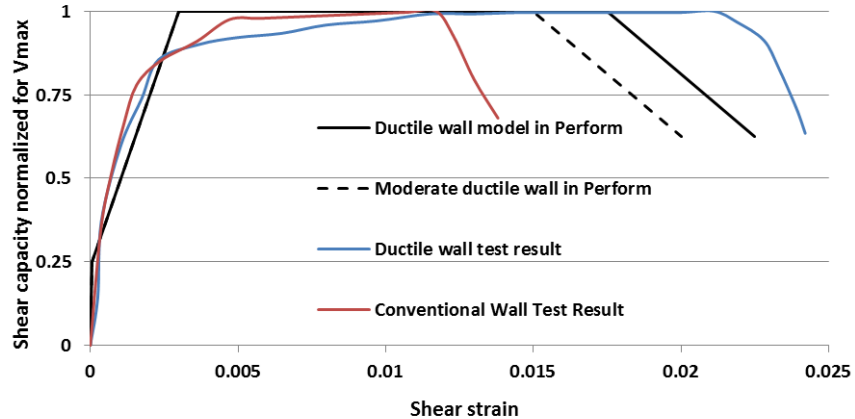


Figure 3-3: Shear material model used in numerical simulation compared with test results (Oesterle 1976)

### 3.1.1. Shear Wall Model Verification

Before using the shear wall models in PERFORM-3D, they were verified against experimental results. This was done by comparing the hysteretic behaviour of analytical shear wall models with those recorded in large-scale wall tests conducted under slowly applied inelastic load reversals. Three test walls with different wall aspect ratios were selected for this purpose. Properties of the shear walls considered for verification are shown in Table. 3-1. The first wall had an aspect ratio of 1.87 (with a 3.0 m height and a 1.6 m length), with a wall thickness of 200 mm. It was tested at the University of Ottawa under reversed cyclic loading (Navidpour and Saatcioglu 2016). The experimental and analytical force-displacement hysteretic relationships are compared in Figure 3-4, and indicate very good correlations. Both relationships showed excellent agreement in strength and deformability up to 2% lateral drift, followed by gradual strength decay. The second wall having an aspect ratio of 1.0 (with a 1.5 m height and a 1.5 m length) with a wall thickness of 100 mm was also tested at the University of Ottawa (Mohammadi-Doostdar 1994). The hysteretic relationships shown in Figure 3-5 once again indicate excellent agreement between experimental and analytical results. The wall performed in a ductile manner up to a displacement ductility ratio of approximately 3.0,

and developed gradual strength decay thereafter, with a higher rate of decay in the pull direction. The third verification was made against a flexural wall having an aspect ratio of 2.4 (with a 4.6 m height and a 1.9 m width), with a wall thickness of 102 mm, tested at the Portland Cement Association (Oesterle 1976). The hysteretic relationships, shown in Figure 3-6, indicate excellent correlations. The wall behaviour in this case was very ductile, showing stable hysteresis loops up to about a displacement ductility ratio of 4.0. Characteristics of the three simulated walls are summarized in Table. 3-1.

The comparisons presented above indicate that the analytical technique used for modelling shear wall elements in PERFORM-3D provide excellent results, as validated by test data for flexure dominant walls under varying degrees of shear stresses, as reflected by the wall aspect ratio.

**Table. 3-1: Properties of shear walls simulated for verification**

	Wall#1	Wall#2	Wall#3
Height(m)	3	1.5	4.6
Length(m)	1.6	1.5	1.9
Thickness(mm)	200	100	102
Length of Boundary Element(mm)	305	300	190
Boundary Element Reinforcement	10#20M	4#10M	4% of Section
Web Longitudinal Reinforcement	2#10M@200mm	2#10M@285mm(0.8%)	-
Web Horizontal Reinforcement	2#15M@175mm	2#10M@250mm(0.8%)	365 kN Shear Capacity
$f'_c$ (Mpa)	82	35	41.4
$F_y$ (Mpa)	550	400	414
$F_u$ (Mpa)	660	700	Strain Hardening Neglected

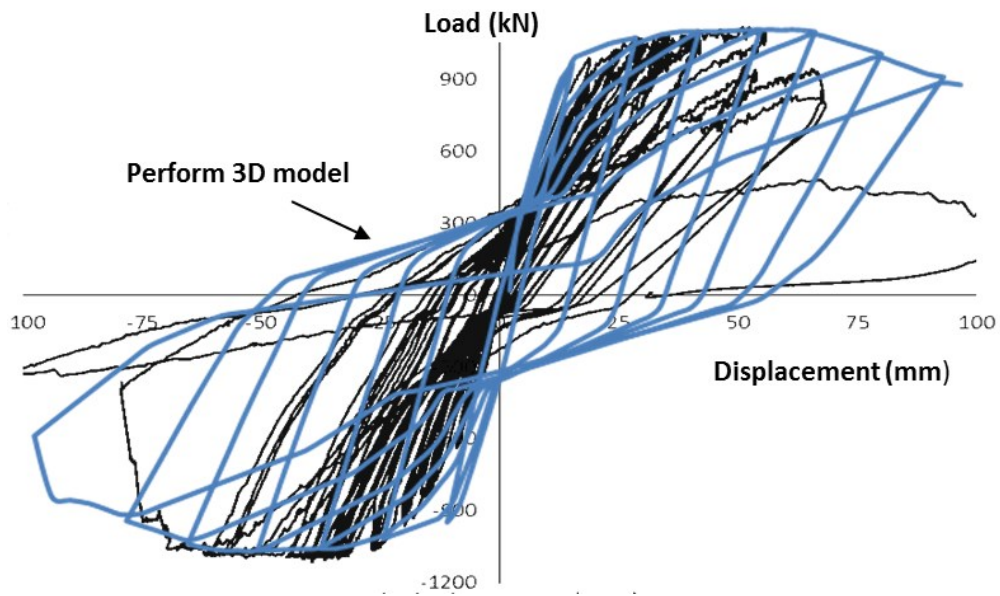


Figure 3-4: Cyclic behaviour of shear wall with aspect ratio of 2 (Navidpour and Saatcioglu 2016)

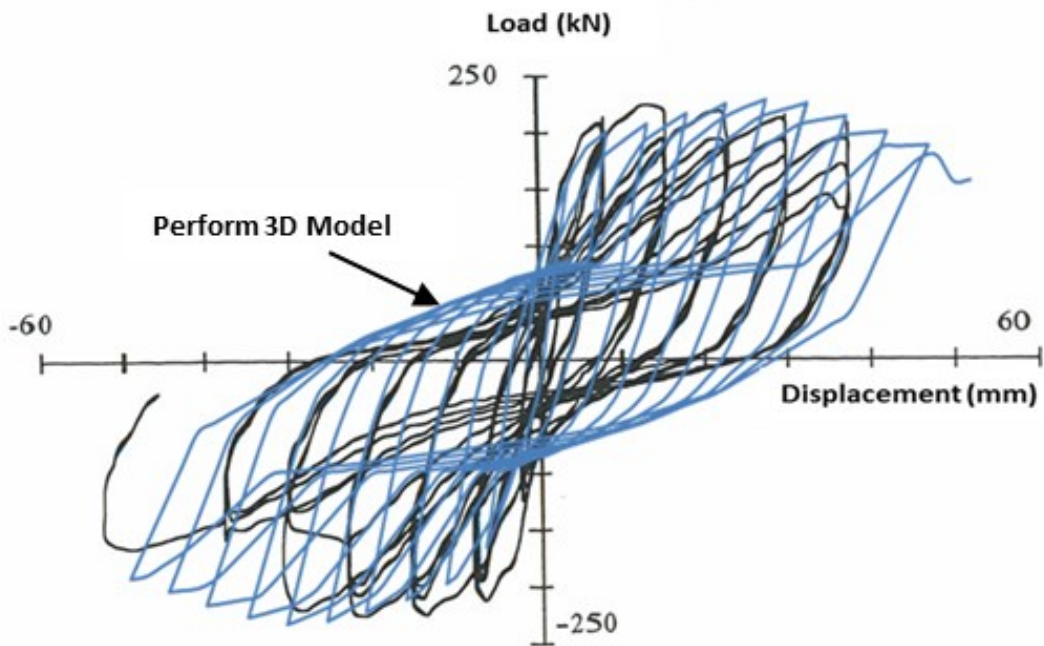


Figure 3-5: Cyclic behaviour of shear wall with aspect ratio of 1 (Mohammadi-Doostdar 1994)

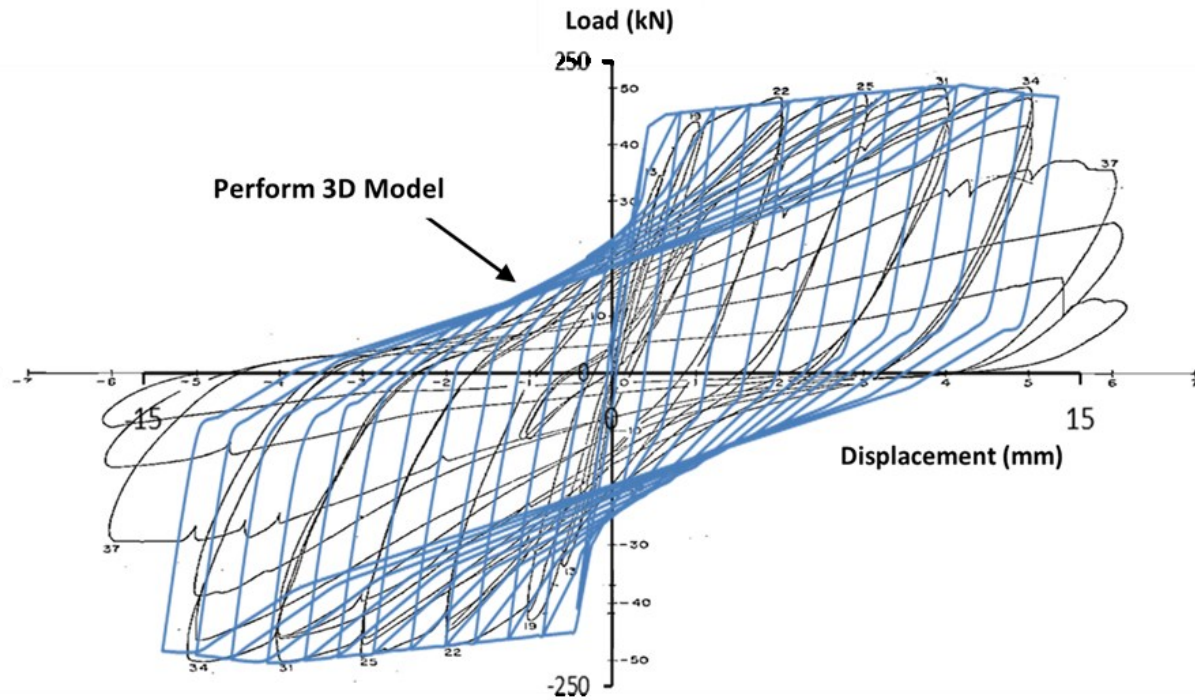


Figure 3-6: Cyclic behaviour of numerical model of wall with aspect ratio > 2 (Oesterle 1976)

Further verification of the shear wall modelling technique was done against analytical results generated by OpenSees software. In this case a 5-storey shear wall was modelled in PERFORM 3D and OpenSees for dynamic inelastic response history analysis. Both computer programs use similar modelling techniques, based on the discretization of wall sections into fibers of concrete and layers of steel at appropriate locations. Material constitutive models were assigned to concrete and steel for sectional analysis. The concrete used in the wall model was unconfined concrete, modelled in OpenSees using the modified Kent and Park model (1971) as implemented by Scott and Fenves (Scott and Fenves 2006), and modelled in PRFORM 3D using Hognestad's concrete model (1951), idealized as five linear segments. The walls were analyzed under the same earthquake record scaled to the peak ground acceleration (PGA) of 1.0g. Figure 3-7 shows the comparison of the displacement time histories obtained from the

use of both software; for the first floor, which developed significant hinging, and for the roof, providing top displacement. The correlation between the results is very good, providing additional verification for the modelling techniques used in PERFORM 3D.

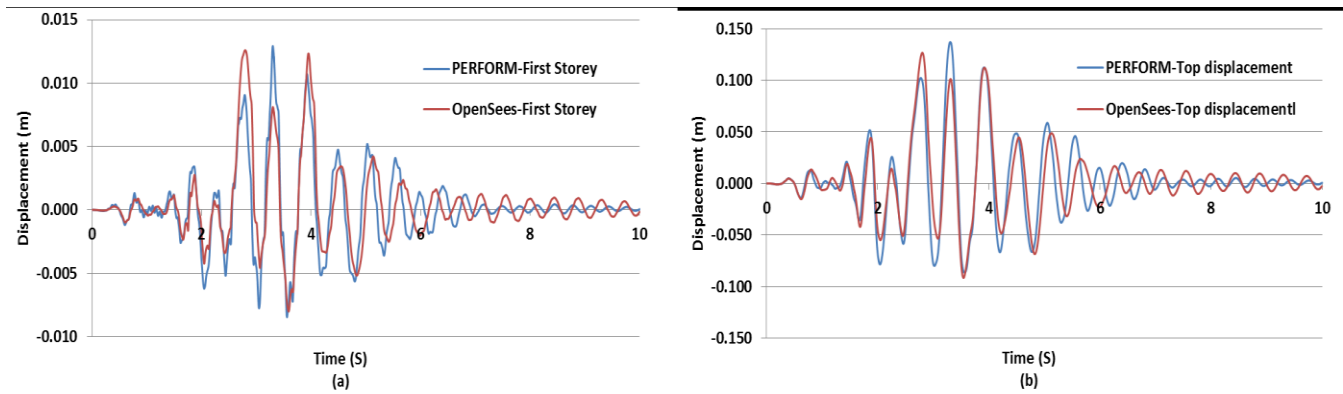


Figure 3-7: Comparisons of displacements for shear walls in PERFORM 3D and OpenSees: (a) First storey and (b) Top displacement

### 3.1.2. Modelling of Frame Elements

The buildings considered for dynamic analysis consisted of two shear walls in each direction with gravity load carrying frames. The frame members were modelled using FEMA Beam and FEMA Column elements that are available in software PERFORM 3D. These elements consisted of elastic components and member end hinges with the point of inflection assumed to occur at member mid-length. This is depicted in Figure 3-8. The elastic component and the hinge together satisfied the required member end moment-chord rotation relationship. It was possible to model the strength degradation in beams and columns, which was essential for modelling the buildings considered in the current research project, as some frame elements could go beyond their capacities under high seismic intensities. Upon the element strength decay, stresses redistributed to adjacent members, resulting in complex behaviour. This behaviour was sensitive to small changes in loads. The analysis ended when too many nonlinear events were detected. This approach is believed to reflect structural response realistically.

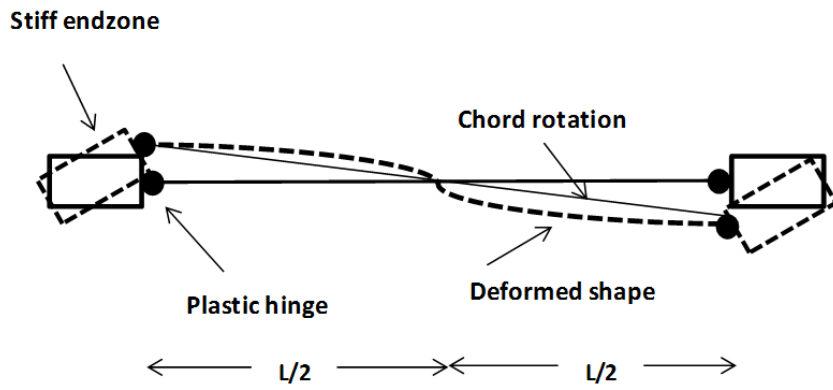


Figure 3-8: FEMA frame element model (CSI 2013)

# **Chapter 4. Fragility Curves for Shear Wall Buildings in Canada Confirming Ductile Design Requirements**

**Abstract:** This paper presents seismic fragility curves and seismic vulnerability assessment of shear wall buildings in Canada designed following the requirements of different versions of the National Building Code of Canada. Two sets of twenty earthquake records compatible with western and eastern Canadian seismicity were selected. Incremental Dynamic Analysis (IDA) was performed by applying selected records with different scale factors to capture structural behaviour under different levels of seismic excitations. Spectral Acceleration and Peak Ground Acceleration were selected as seismic intensity parameters and the first storey drift ratio was selected as engineering demand parameter. The results of IDA were used to generate seismic fragility curves for selected buildings as probabilistic tools to assess seismic behaviour of reinforced concrete shear walls in Canada. The resulting curves depict the probability of exceeding pre-determined limit states for performance-based assessment under different levels of seismic intensity. Selected performance levels consist of; i) immediate occupancy, ii) life safety and iii) collapse prevention. The fragility curves were

developed for 2-storey, 5-storey and 10-storey buildings with regular floor plans, covering a wide range of building periods used in practice. The buildings were designed following the requirements of the 2010 NBCC for Vancouver and Ottawa. Their applicability to buildings designed after the enactment of modern seismic codes in Canada, dating back to 1975, is discussed. The structures were simulated with 3D modelling, and their seismic vulnerability was assessed through the seismic fragility curves generated. The results show that seismic code conforming buildings designed based on recent NBCC satisfy the objective of life safety at seismic events with 2475-year return period.

#### **4.1. Introduction**

Structural damage observed after past earthquakes has resulted in the development of improved seismic design provisions in modern building codes. The main objective of seismic design is to provide life safety during strong earthquakes, limit building damage during low to moderate earthquakes, and for post-disaster buildings to continue maintaining their functions during and after the earthquake with minimal damage. These requirements, while have been effectively used for the design of new buildings, fall short of providing the level of acceptable risk if seismic retrofitting becomes a consideration. A probabilistic methodology for assessing seismic vulnerability of existing buildings is best suited to earthquake investigations, with capabilities for providing support to decision makers.

Seismic vulnerability assessment of buildings can be done by conducting inelastic time history analysis, which provides information on structural response of critical elements. This may be justifiable for a few important buildings, but may be costly and time consuming if a large building inventory is required to be assessed. Fragility curves provide convenient tools for seismic vulnerability analysis of buildings having similar characteristics. Therefore, their use as seismic assessment tool becomes feasible if the

buildings are classified into groups based on their structural and geometric properties. The overall objective of the current research program at the University of Ottawa is to develop such tools for reinforced concrete and masonry buildings having different building heights and corresponding fundamental periods, designed for different eras, with or without irregularities. This paper is devoted to fragility analysis of reinforced concrete shear wall buildings designed in Canada following post 1975 practice and having regular structural layouts. Similar research has been conducted in the US and Europe for seismic assessment of code conforming RC frame buildings (Haselton *et al.* 2010).

Seismic vulnerability of buildings varies based on the seismicity of the region. In Canada, two regions show seismic characteristics that are different. The tectonic structure of western Canada includes active faults, consists of both a mega subduction zone and a number of strike-slip faults, potentially resulting in strong earthquakes. The eastern Canadian earthquakes are intra-plate events attributed to the weaknesses of the crust, generally resulting in low to moderate intensity earthquakes. For seismic risk assessment in Canada it is important to consider these differences in seismicity. Therefore, two cities were considered in generating the fragility curves; Vancouver located in the west and Ottawa located in the east. Three different building heights were considered for each city; 2-Storey, 5-Storey and 10-Storey. A set of 20 synthetic records, developed for the region, was used for each region. The buildings in Vancouver were designed following the ductile design and detailing requirements of CSA A23.3-04 (CSA 2004), while the buildings in Ottawa were designed as moderately ductile buildings, following the requirements of the same standard.

Seismic building code development in Canada can be categorized into two broadly defined eras. The design practice prior to the 1975 National Building Code of Canada (NRCC 1975) did not consider ductile design and associated energy dissipation in buildings. Seismic design forces were defined as percentage of seismic mass, with little

considerations given to dynamic characteristics of buildings and the intricacies of Canadian seismic hazard values. Buildings designed and built in this era are potentially vulnerable to strong earthquakes. They are addressed in Chapter 5 and 6, including the effects of irregularities in such buildings. The fragility curves in this chapter were developed for buildings conforming to the 2010 NBCC, but can be used approximately for buildings designed since 1975 with some empirical modifications and consideration as explained in the following section.

## 4.2. Selection and Design of Buildings

Six shear wall buildings with three different heights (2-storey, 5-storey and 10-storey) were selected for design. The buildings had the same regular floor plan as illustrated in Figure 4-1. Three buildings were designed for Vancouver and the other three companion buildings were designed for Ottawa using the seismic design requirements of NBCC2010 (NRCC 2010) and CSA A.23 (CSA 2004). The buildings in Vancouver were designed to be fully ductile, consistent with a ductility related force modification factor of  $R_d = 3.5$ . This implies that the shear wall boundary elements were fully confined following the requirements for ductile column confinement. The impact of this requirement on shear wall design has remained essentially the same since the 1975 NBCC, delaying the onset of strength decay in the analytical model for hysteretic relationship. The buildings in Ottawa were designed to be moderately ductile, consistent with  $R_d = 2.0$ . The concrete used in all buildings was normal density concrete ( $w_c = 2400 \text{ kg/m}^3$ ) with a cylinder strength of  $f'_c = 35 \text{ MPa}$ . The grade of reinforcing steel used was  $f_y = 400 \text{ MPa}$ . The design live load (L) was equal to 1.9 kPa, as specified in NBCC2010 for residential buildings with 1.0 kPa live load for the roof. The design dead load included 1.0 kPa of superimposed dead load, over and above the weight of the structure. All the buildings were assumed to be built on firm soil or soil of class “C” and corresponding acceleration and velocity based soil classification coefficients of  $F_a = F_v = 1.0$ . For regular structures

with less than 60 m height and fundamental period less than 2.0 sec in each of the two orthogonal directions, NBCC allows the use of the Equivalent Static Force Procedure for computing earthquake design forces. The Uniform Hazard Spectra (UHS) values,  $S_a(T)$  specified in NBCC2010 for 2% probability of exceedance in 50 years, were used in design. In western Canada, Vancouver has the second highest seismicity level after Victoria among the major cities, representing high seismicity. In eastern Canada, Montreal and Ottawa have similar Spectral Acceleration values, corresponding to moderate seismicity.

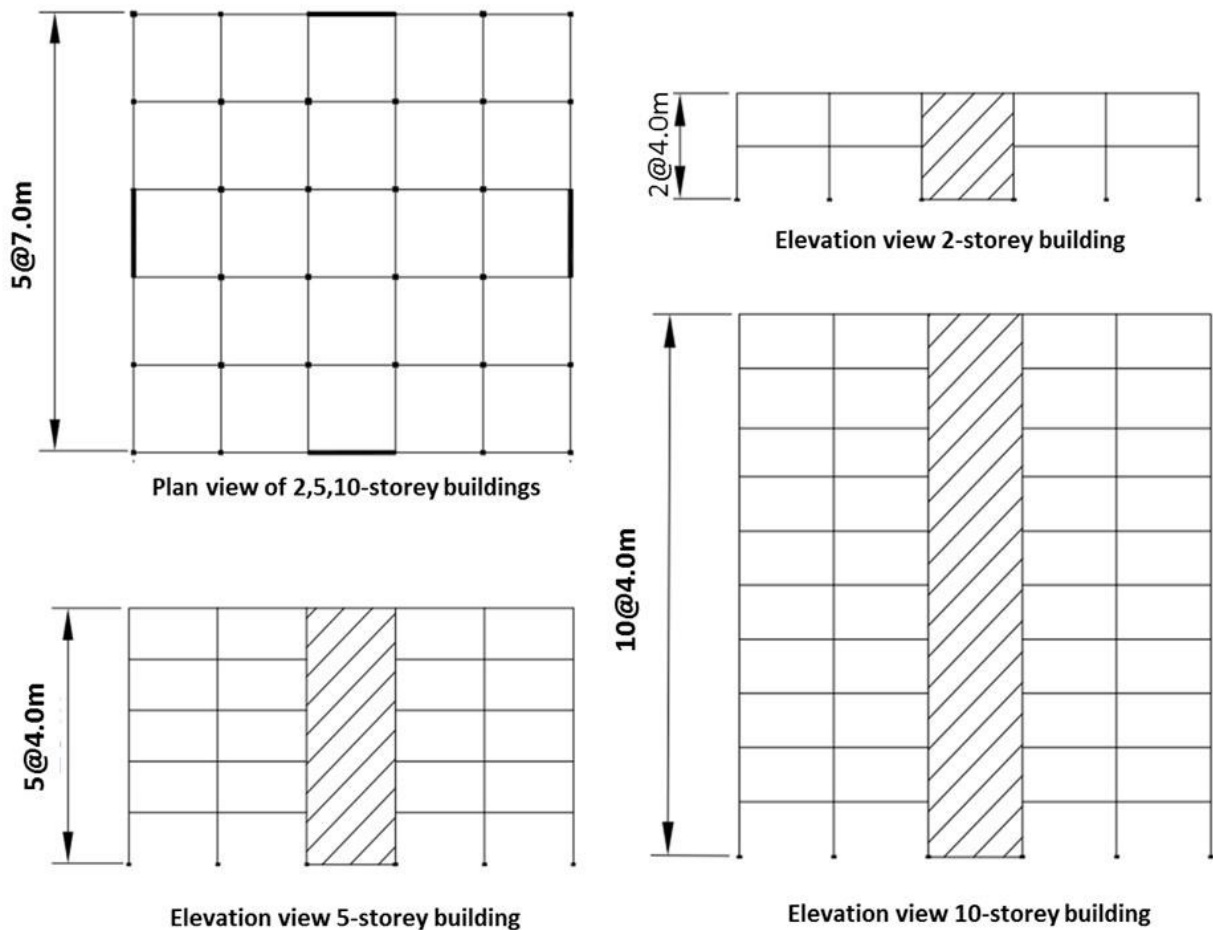


Figure 4-1: Geometry of selected buildings

The structural period was calculated by the empirical equation specified in the 2010 NBCC,  $T_a = 0.05(h_n)^{3/4}$ . However, the code allows the use of the building period computed based on the analytical model of the building and its dynamic response, provided that this value does not exceed the empirical value by a factor of 2.0. The dynamic periods were used in design within the limitation stated in the code, as shown in Table 4-1. The equivalent elastic static base shear  $V_{ed}$  is obtained as the product of the spectral acceleration at the design period, soil modification factor, importance factor  $I_E$ , and the structural mass. The inelastic design base shear  $V_d$  is then obtained by dividing  $V_{ed}$  by the ductility and over-strength related force modification factors,  $R_d$  and  $R_o$  ( $R_d=3.5$ ,  $R_o=1.6$  for Vancouver;  $R_d=2.0$ ,  $R_o=1.4$  for Ottawa; and  $R_d=2.0$ ,  $R_o=1.4$  for squat walls with aspect ratios of less than 2.0 for both cities). Table 4-1 provides the period and design base shear factor (base shear/weight) for each building.

**Table 4-1: Design Periods and Base Shear Factors**

	Code Period	Max. Dynamic Period	Modal Period(S)	Perform 3D	Design Period(S)	Dynamic Base Shear Factor			
						Sa(T) Vancouver	Sa(T) Ottawa	Vancouver	Ottawa
5-Storey	0.473	0.946	First Lateral Mode	0.816	0.816	0.46	0.21	0.082	0.076
			First Torsional Mode	0.476					
			Second Lateral Mode	0.423					
2-Storey	0.238	0.476	First Lateral Mode	0.530	0.476	0.65	0.32	0.232	0.114
			First Torsional Mode	0.286					
			Second Lateral Mode	0.063					
10-Storey	0.795	1.591	First Lateral Mode	1.721	1.591	0.24	0.085	0.054	0.051
			First Torsional Mode	1.215					
			Second Lateral Mode	0.558					

The walls were required to be designed and detailed for plastic hinges over a height equal to at least 1.5 times the length of the longest wall above the design critical section. A slightly longer plastic hinge region was considered in design, covering the bottom three storeys for 5-storey and 10-storey buildings, and the bottom two storeys for the 2-storey buildings. The flexural and shear reinforcement requirements for the critical section were maintained over the plastic hinge length. For all elevations above the plastic hinge region, the design overturning moments were increased by the ratio of the factored

moment resistance to the factored moment, both calculated at the top of the plastic hinge region. It was insured that the factored shear resistance of the walls was greater than the larger of the effects of factored loads and shear forces associated with flexural capacities. The flexural capacities used for this purpose were probable moment resistances for ductile walls and nominal flexural resistances for moderately ductile walls. For the latter case, the shear demand computed was taken as being not greater than that corresponding to factored loads with  $R_d R_o$  equal to 1.0. The magnification of shear due to inelastic effects of higher modes was also considered.

The factored shear resistance of walls was computed as the summation of  $V_c$  and  $V_s$  accounting for the contributions of concrete and shear reinforcement, respectively.  $V_c$  and  $V_s$  were calculated based on Equations 4.1 and 4.2, also given in CSA A23.3 (CSA 2004).

$$V_c = \phi_c \lambda \beta \sqrt{f'_c} b_w d_v \quad \text{Eq. 4.1}$$

$$V_s = \frac{\phi_s A_v d_v \cot \theta}{s} \quad \text{Eq. 4.2}$$

where  $\phi_c$ ,  $\phi_s$  are the resistant factors for concrete and reinforcing steel, respectively.  $b_w$  is the web width,  $d_v$  is the effective shear depth, and  $A_v$  is the area of shear reinforcement within distance  $s$  and  $\theta$  is assumed as  $42^\circ$ .

### 4.3. Numerical Modelling and Dynamic Analysis

#### 4.3.1. Software PERFORM-3D and Numerical Building Models

Dynamic inelastic response history analyses were performed using the computer software PERFORM-3D (CSI 2013). The software was developed by Computers and Structures, Inc. (CSI) for Nonlinear Analysis and Performance Assessment of 3D Structures. Perform 3D has variety of options for nonlinear modelling of elements including plastic hinge models and fiber models. The fiber model is built by discretizing element sections into strips and assigning material constitutive relationships to each strip. Hence, it is believed to provide a more accurate representation of member behaviour than the plastic hinge models, which lumps plastic deformation at member ends. The response of shear wall buildings is mainly dominated by the behaviour of shear walls, and the wall response plays a major role in overall building response. Therefore, fiber-discretized modelling was employed to simulate the wall behaviour. The overall behaviour of a shear wall section in a fiber model is obtained from the integration of the effects of concrete and steel materials, with due considerations given to the confinement of concrete. The confined concrete model proposed by Saatcioglu and Razvi (1992) was used for the wall boundary elements and unconfined concrete by Hognestad (1951) was used for the rest of the wall section. The numerical modeling techniques used for 2-storey, 5-storey and 10-storey buildings for PERFORM-3D were discussed in Chapter 3. The models were verified against tests of large-scale shear walls with different aspect ratios, tested under simulated seismic loading. The modelling was further verified against the results of analyses of walls based on similar models, using OpenSees as the software.

PERFORM-3D enables modelling of frame elements by means of FEMA Beam and FEMA Column components. These components consist of two elements, where each element consists of an elastic segment and member end hinge between the member end and the point of inflection. Each element represents the appropriate end moment-end

rotation relationship. It is also possible to model strength loss for beams and columns, which is an essential characteristic for modelling components in the current research, as the level of applied incremental load can go beyond the capacity for some elements. Upon experiencing strength loss, the lost strength is redistributed to the adjacent components, resulting in a complex behaviour, sensitive to small changes in loads. The program stops the analysis when a large number of nonlinear events take place. The numerical models for the buildings analyzed are shown in Figure 4-2.

#### 4.3.2. Incremental Dynamic Analysis

The development of fragility curves for seismic vulnerability assessment involves a large number of response history analyses under incrementally changing earthquake intensity. This can best be handled through Incremental Dynamic Analyses (Vamvatsikos and Cornell 2002a). Incremental Dynamic Analyses (IDA) is a powerful method of estimating structural performance under different levels of seismic excitation by subjecting the structural model to a set of ground motion records, each scaled to multiple levels of intensity. The results are represented in the format of IDA curves showing the variation of a response parameter with seismic intensity level. Choosing several records compatible with the site condition enables probabilistic analyses of IDA, which can be presented in the format of seismic fragility curves. IDA gives thorough understanding of the range of structural response within the range of potential ground motion intensities. It also provides structural behaviour under severe levels of ground motion until failure, in addition to providing the estimation of dynamic capacity of the global structural system.

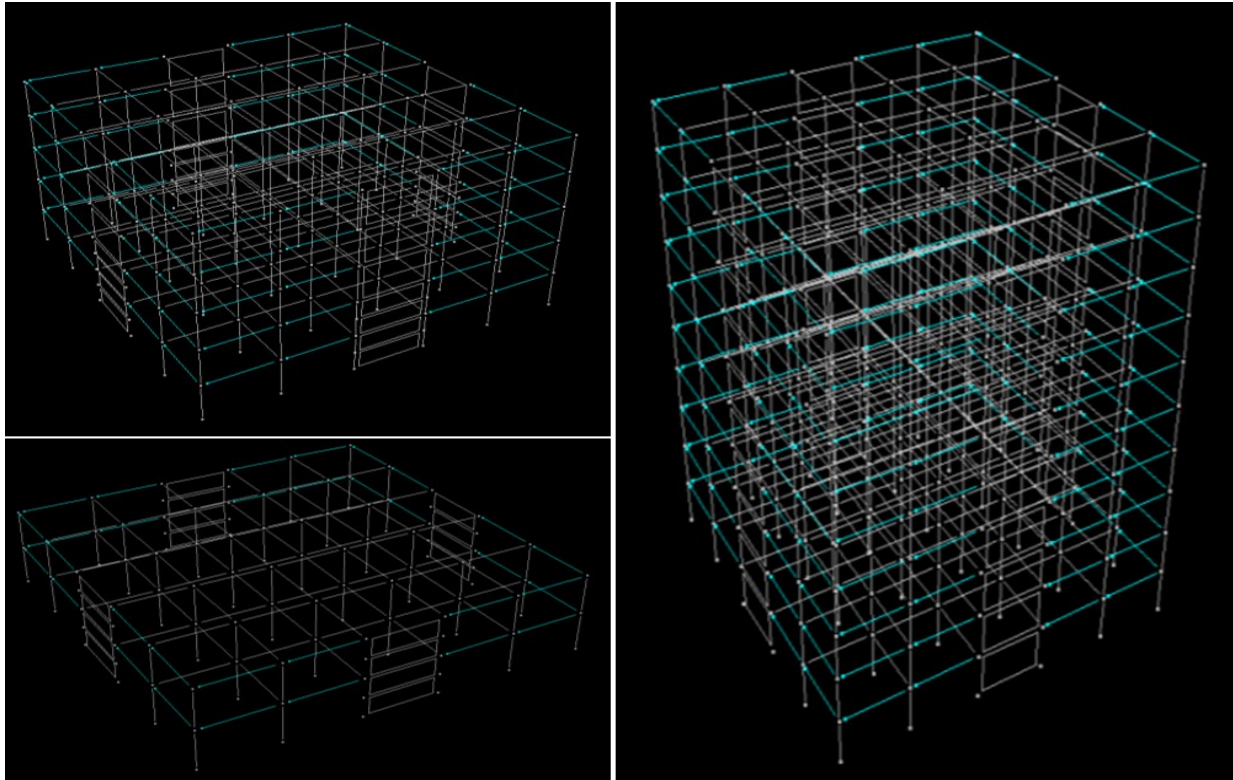


Figure 4-2: Numerical simulation of buildings in PERFORM-3D

#### 4.3.3. Earthquake Record Selection and Scaling

In order to perform IDA, a set of ground motion records representative of the building site is needed. Herein, artificial earthquake ground motions, generated by Atkinson (2009), were used. These records are compatible with the uniform hazard spectra (UHS) specified for seismic design in the 2005 and 2010 editions of the National Building Code of Canada for earthquakes having 2% probability of exceedance in 50 years. The "target" UHS depends on the location and the site condition, where the site condition is classified based on the time-averaged shear-wave velocity in the top 30 m of soil deposit (soil types A, C, D, and E specified in the building code). Atkinson applied the stochastic finite-fault method to generate earthquake time histories that may be used to match the 2005 NBCC UHS for a range of Canadian sites and different soil types. In this study, the records generated for the reference soil type C were used.

The records were provided in four sets of 45 time histories, where each set corresponded to a different magnitude (M) distance combination: M6.5 at 10 to 15 km, M6.5 at 20 to 30 km, M7.5 at 15 to 25 km, and M7.5 at 50 to 100 km for Western Canada and four sets of 45 time histories: M6 at 10 to 15 km, M6 at 20 to 30 km, M7 at 15 to 25 km, and M7 at 50 to 100 km for Eastern Canada. Five records were selected from each of these eight sets of records, resulting in twenty records for each site, which were scaled to match the target spectrum in the period range of 0.5 to 2.5 for eastern and western Canada. These records were selected to have the lowest standard deviation for the ratio of simulated response spectra to the target UHS  $(S_a)_{\text{target}}/(S_a)_{\text{simulated}}$  in the range of periods of interests. Figure 4-3 shows the comparison of the response spectra for the twenty records selected and the UHS for Vancouver. For IDA, the selected ground motion records needed to be scaled to cover the entire range of structural response, from elastic behaviour to yield, and from yield to structural failure.

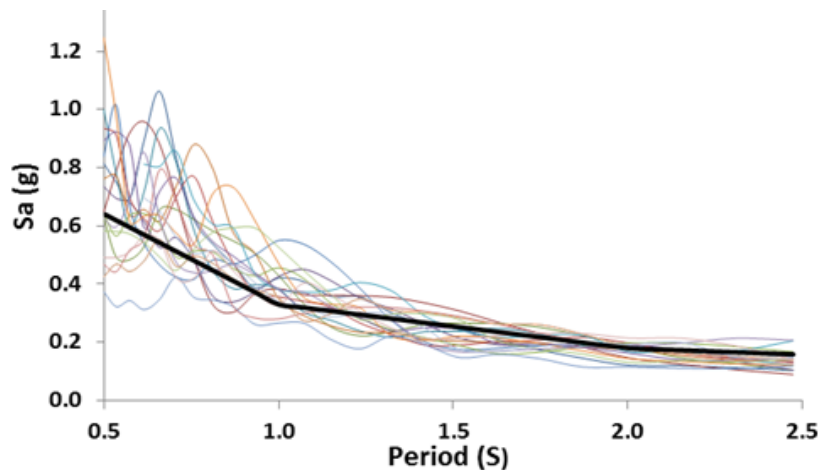


Figure 4-3: Comparison of UHS and spectral accelerations of selected records for Vancouver

#### 4.3.4. Seismic Intensity Measure and Engineering Demand Parameter

Incremental dynamic analysis involves selecting two parameters: i) seismic intensity measure and ii) engineering demand parameter. Two seismic intensity measures were selected in this study, 5% damped spectral acceleration at fundamental period ( $S_a(t_1)$ ),

5%) and peak ground acceleration (PGA).  $S_a(t_1)$  was used by previous researches, including Vamvatsikos and Cornell (2002a) and Ellingwood *et al.* (2007). This measure of intensity reflects the characteristic of the earthquake, while also dependent on the structural period, as opposed to Peak Ground Acceleration (PGA), which only reflects the characteristic of the earthquake record. Moreover,  $S_a$  is a parameter defined in the National Building Code of Canada as a design parameter, and frequently used by designers. On the other hand, scaling based on PGA is easier and gives a uniform format of hazard for structures with different period, allowing the comparison of buildings with different heights or periods.

The engineering demand parameter (EDP) is also needed for IDA to represent the damage state of the building. Herein, the first storey horizontal drift ratio was selected as EDP, which indicates the level of damage. The use of inter-storey drift to define different limit states is quite common among engineers as it can be computed and rationalized easily. Furthermore, the first storey generally experiences higher level of plastic deformations under predominantly flexural response, with storey drift becoming a preferred EDP.

#### 4.3.5. IDA Results

The use of a set of twenty records with different scale factors resulted in approximately 200 dynamic analyses for each building model. The resulting IDA curves based on spectral acceleration are shown in Figure 4-4 for the ductile buildings designed for Vancouver, subjected to western seismicity, and moderately ductile buildings designed for Ottawa, subjected to eastern seismicity. The IDA results based on PGA were also obtained and presented in Figure 4-5, in terms of PGA versus inter-storey drift using logarithmic scale. The results in the latter set are more dispersed for Ottawa than those for Vancouver. This is caused by time history records provided for short duration

earthquakes in the east, which result in lower levels of damage when scaled based on PGA.

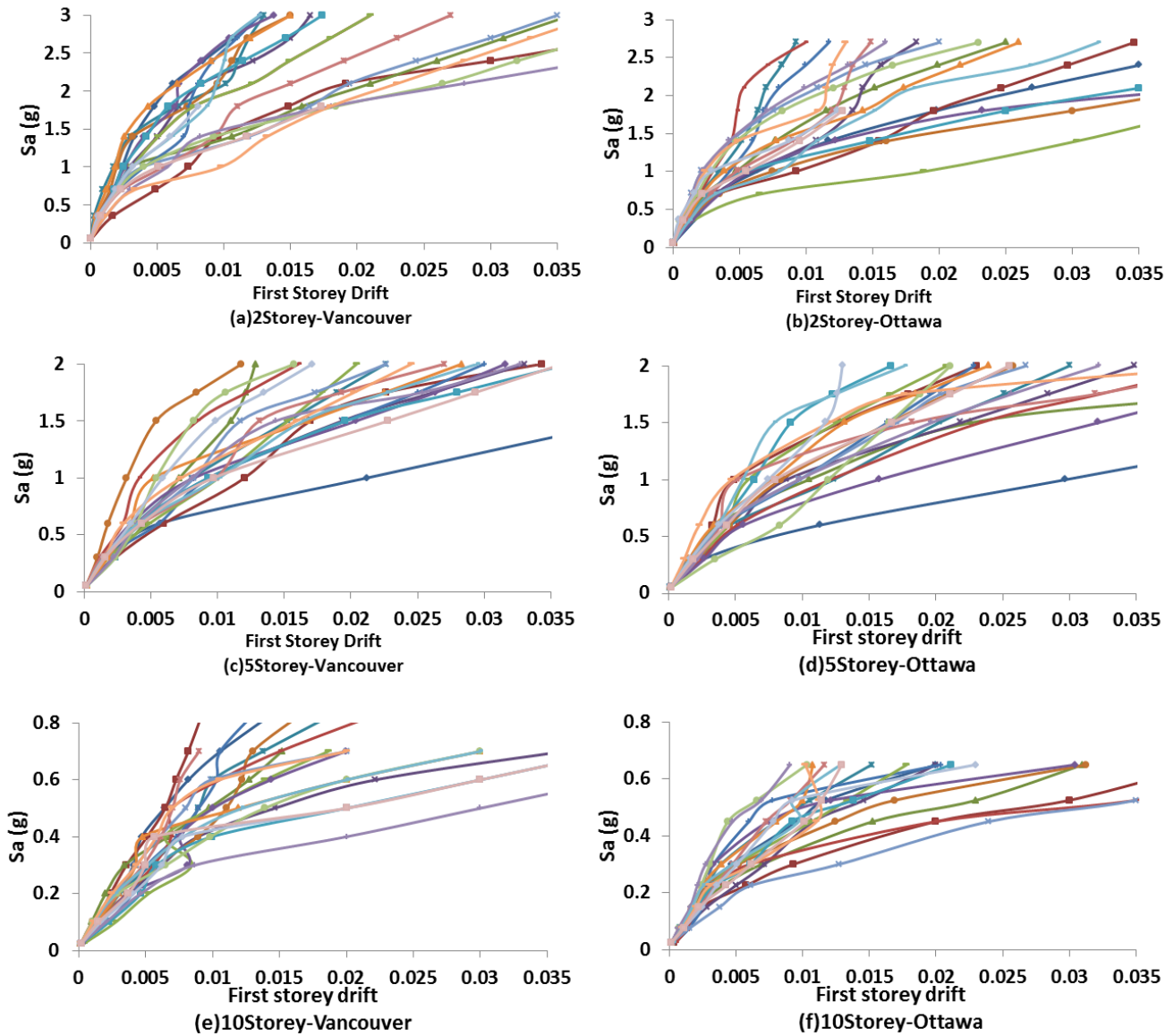


Figure 4-4: Result of IDA for 2-, 5-, and 10-storey buildings

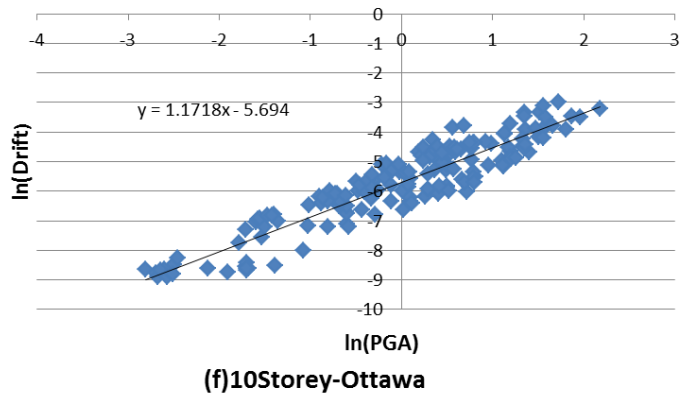
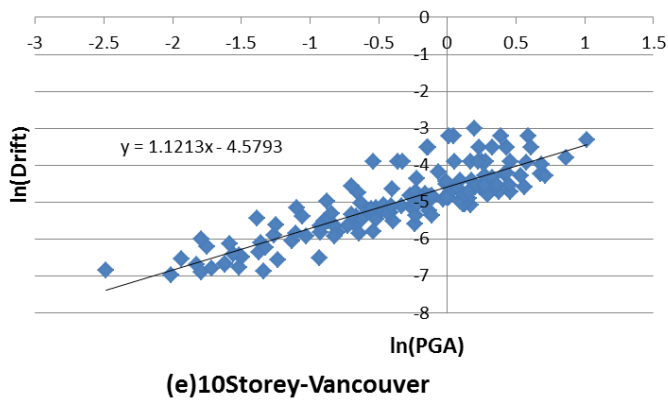
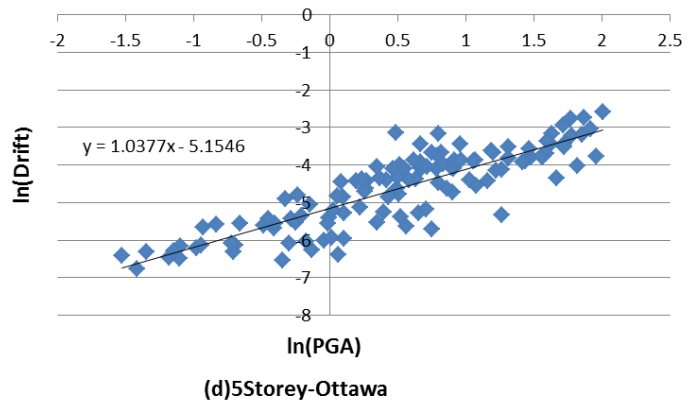
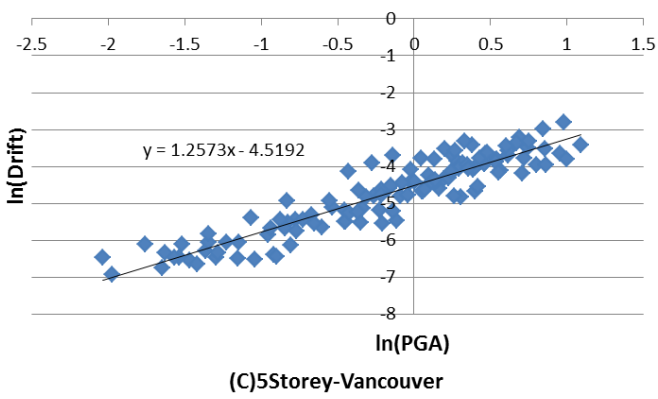
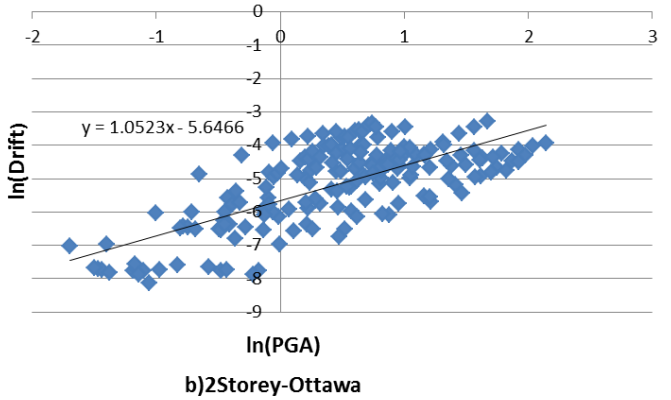
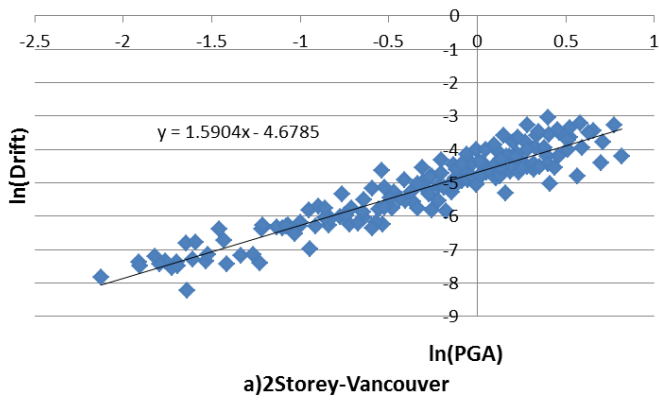


Figure 4-5: PGA-drift relationships for 2-, 5- and 10-storey buildings

## 4.4. Development of Fragility Curves

Seismic fragility describes the probability of reaching or exceeding predefined limit states as a function of a severity of ground motion intensity considering uncertainties associated with structural capacity and earthquake characteristics. Damage data required to develop fragility curves can be obtained through post damage observation, experimental results, analytical modelling and a combination of these methods. As there are not enough observational or experimental results for most sites, especially for Canadian cities, use of analytically-simulated structural damage statistics is the most practical option.

### 4.4.1. Damage Limit States

The IDA curves presented in the preceding section quantify structural response in terms of first storey drift ratios. However, limit states for lateral drift need to be defined to assess the level of damage in a building. Three different limit states have been adopted to express the state of damage in buildings: i) Immediate Occupancy, ii) Life Safety and iii) Collapse Prevention. These limit states are the same as those described in ASCE 41(ASCE 2007) as follows:

- *Immediate occupancy*: Building remains safe to be reoccupied; lateral-force and gravity-load-resisting systems retain most of their design strengths.
- *Life safety*: Significant damage has occurred to the structure; structural elements and components may be severely damaged; gravity-load-carrying elements continue functioning.
- *Collapse prevention*: Substantial damage has occurred to structural elements; significant strength and stiffness degradation of the lateral-load-resisting system has taken place; large permanent lateral deformations were induced to the structure; the structure is not repairable and not safe to reoccupy.

The above provide qualitative descriptions of damage states, while quantitative values are required to use the results of IDA. Following the ASCE 41 recommendation, 0.5% storey drift ratio was selected as the threshold for immediate occupancy. Observations made from the IDA curves confirm that until this drift value is exceeded the structure remains essentially elastic, allowing the building to be immediately re-occupied. The life safety limit state was adopted from ASCE 41 as 1% drift of the first storey. The analysis results indicated that the buildings remained intact at this drift level, while experiencing some limited inelasticity. For the collapse prevention limit state, ASCE 41 suggests a value of 2% drift for shear wall buildings. However, the collapse state indicated by the response history analysis conducted in the current investigation made it necessary to re-assess this limit. The collapse was defined in the current study as either the onset of numerical instability in the structure which is triggered by a large number of elements experiencing inelasticity in a given time step, or significant softening of the buildings as indicated by the reduction in the post-yield slope of the IDA curve to drop below 20% of the effective elastic slope, whichever occurred first. The same definition was adopted previously by Jeong and Elnashai (Jeong and Elnashai 2007) and can be rationalized as it indicates a significant change in drift with a small change in earthquake intensity. A wide range of drift ratios were observed at collapse under different earthquake records. Drift values corresponding to observed collapse of structures under different records are shown in Table 4-2. For each case the median value presented in the table is assumed as limit state. Figure 4-6 to Figure 4-8 show the fragility curves for the 2, 5 and 10-storey shear wall building designed on the basis of the NBCC 2010 for fully ductile buildings in Vancouver and moderately ductile buildings in Ottawa. Fragility curves were also derived using PGA as seismic hazard level for 2-storey, 5-storey and 10-storey buildings, as shown in Figure 4-9 to Figure 4-11, respectively.

#### 4.4.2. Fragility Curves

Analytical fragility curves were derived from the results of IDA described earlier, expressing the probability of reaching or exceeding predefined limit states as a function of the severity of ground motion intensity. The fragility function is described in equation 4.1 (FEMA-273).

**Table 4-2: Observation of collapse condition under different records**

Earthquake	Observed Drift at Collapse-Vancouver			Earthquake	Observed Drift at Collapse-Ottawa		
	2-Storey	5-Storey	10-Storey		2-Storey	5-Storey	10-Storey
<b>W1</b>	0.012	0.022	0.016	<b>E1</b>	0.012	0.020	0.016
<b>W2</b>	0.013	0.014	0.014	<b>E2</b>	0.013	0.017	0.015
<b>W3</b>	0.017	0.011	0.012	<b>E3</b>	0.017	0.028	0.019
<b>W4</b>	0.017	0.015	0.015	<b>E4</b>	0.017	0.030	0.017
<b>W5</b>	0.014	0.017	0.015	<b>E5</b>	0.014	0.016	0.020
<b>W6</b>	0.020	0.015	0.015	<b>E6</b>	0.020	0.031	0.030
<b>W7</b>	0.015	0.025	0.021	<b>E7</b>	0.015	0.016	0.020
<b>W8</b>	0.016	0.011	0.011	<b>E8</b>	0.016	0.016	0.013
<b>W9</b>	0.012	0.015	0.014	<b>E9</b>	0.012	0.020	0.020
<b>W10</b>	0.015	0.023	0.020	<b>E10</b>	0.015	0.022	0.030
<b>W11</b>	0.020	0.017	0.017	<b>E11</b>	0.017	0.015	0.020
<b>W12</b>	0.017	0.019	0.016	<b>E12</b>	0.017	0.017	0.020
<b>W13</b>	0.025	0.032	0.015	<b>E13</b>	0.025	0.017	0.030
<b>W14</b>	0.018	0.020	0.018	<b>E14</b>	0.018	0.033	0.015
<b>W15</b>	0.020	0.013	0.012	<b>E15</b>	0.020	0.013	0.015
<b>W16</b>	0.021	0.015	0.013	<b>E16</b>	0.021	0.037	0.015
<b>W17</b>	0.015	0.017	0.017	<b>E17</b>	0.017	0.020	0.015
<b>W18</b>	0.015	0.014	0.012	<b>E18</b>	0.023	0.020	0.015
<b>W19</b>	0.015	0.013	0.012	<b>E19</b>	0.017	0.013	0.025
<b>W20</b>	0.025	0.019	0.016	<b>E20</b>	0.025	0.021	0.015
<b>Median</b>	<b>0.017</b>	<b>0.016</b>	<b>0.015</b>	<b>Median</b>	<b>0.017</b>	<b>0.02</b>	<b>0.018</b>

$$P[D > Di | IM] = \phi \left[ \frac{\ln(x / Di)}{\sqrt{\beta_{d/IM}^2 + \beta_c^2 + \beta_m^2}} \right] \quad \text{Eq. 4.1}$$

in which  $\phi(\cdot)$  is the standard normal cumulative distribution function,  $D_i$  is the upper bound for each damage level,  $x$  is the median value of demand as a function of IM.  $\beta_{d/IM}$  is the dispersion (logarithmic standard deviation) of the demand conditioned on the IM derived based on dispersion of IDA results.  $\beta_c$  is the capacity uncertainty assumed 0.3 based on recommendation of Wen *et al.* (2004) and  $\beta_m$  is the modeling uncertainty assumed 0.2 following Celik and Ellingwood (2010) based on the assumption that response of the numerical models falls in 30% difference with 90% confidence bound)

IDA is a powerful analysis method, which provides input in a probabilistic framework to estimate the annual likelihood of the event that the demand exceeds a pre-defined limit-state. Using the results of IDA, probability of exceeding limit states of immediate occupancy, life safety and collapse prevention defined above were calculated for different levels of spectral accelerations ( $S_a(T_1)$ ) or for different PGAs. Figure 4-6 to Figure 4-8 show the fragility curves for the 2, 5 and 10-storey shear wall building designed on the basis of the NBCC 2010 for fully ductile buildings in Vancouver and moderately ductile buildings in Ottawa. Fragility curves were also derived using PGA as seismic hazard level for 2-storey, 5-storey and 10-storey buildings, as shown in Figure 4-9 to Figure 4-11, respectively.

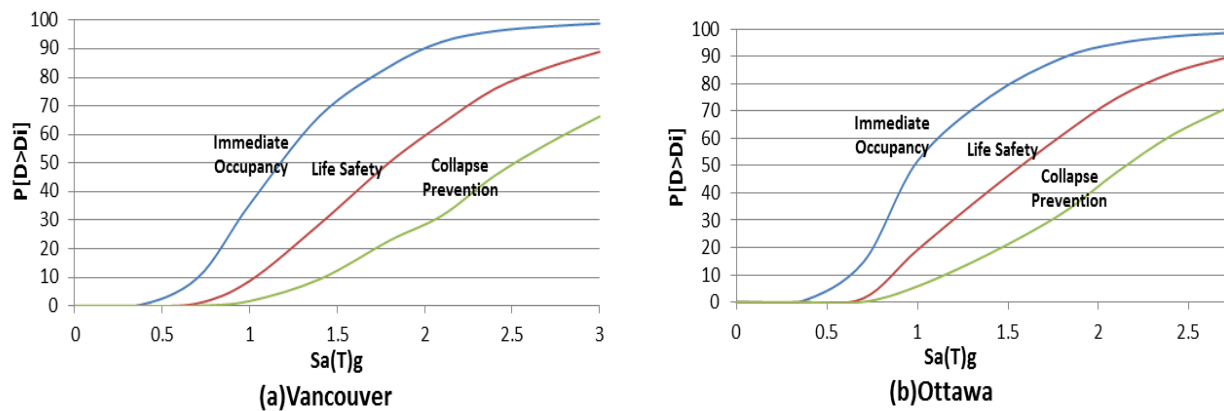


Figure 4-6: Fragility curves derived for 2-Storey shear wall buildings based on spectral acceleration

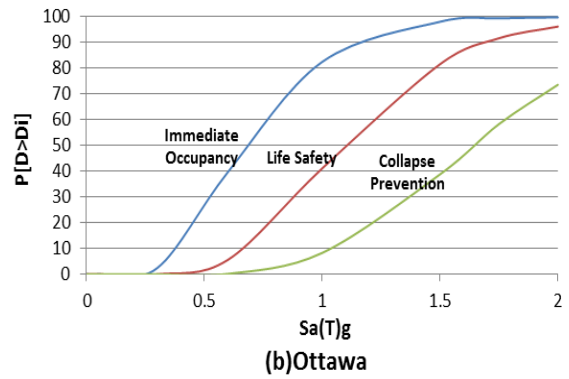
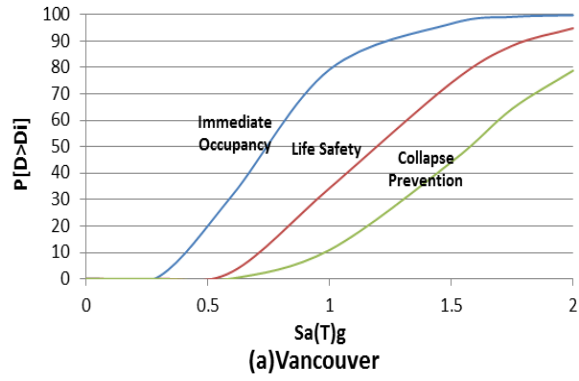


Figure 4-7: Fragility curves derived for 5-Storey shear wall building based on spectral acceleration

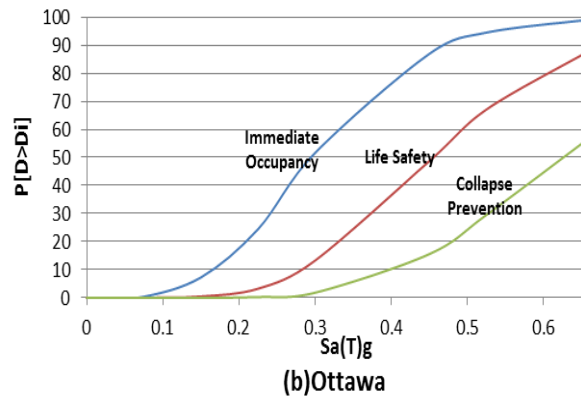
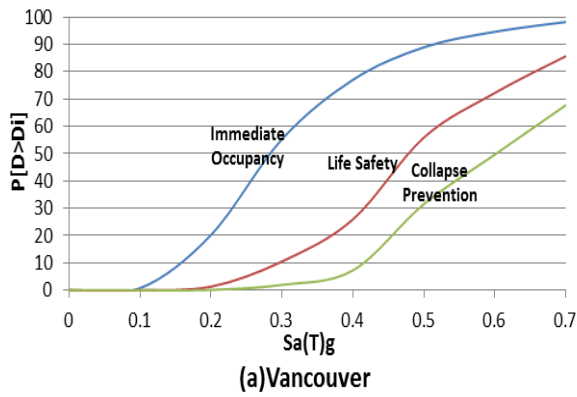


Figure 4-8: Fragility curves derived for 10-Storey shear wall building based on spectral acceleration

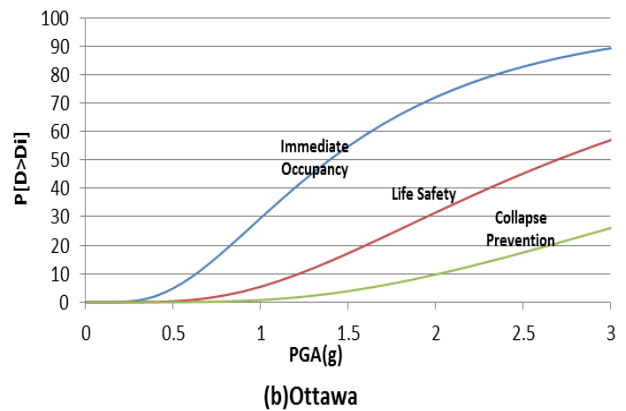
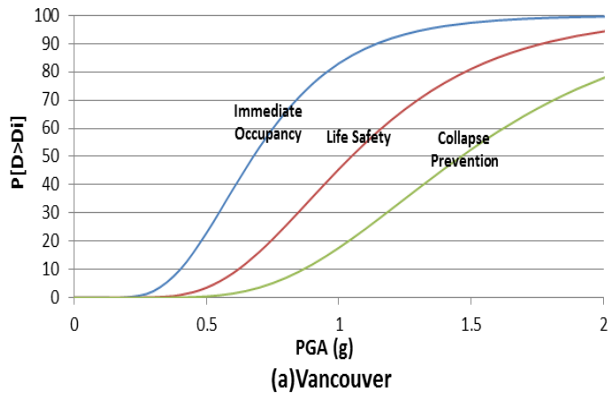


Figure 4-9: Fragility curves derived for 2-Storey shear wall building based on Peak Ground Acceleration

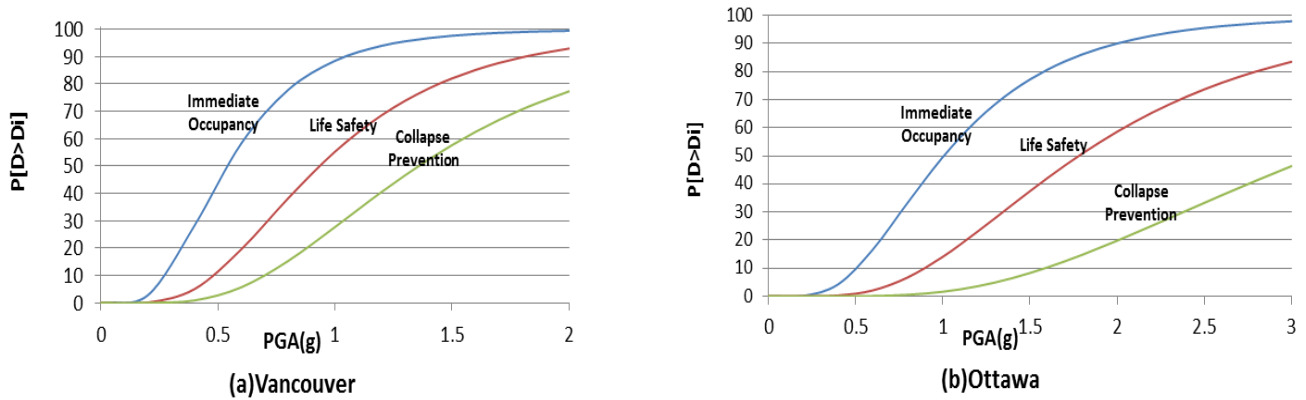


Figure 4-10: Fragility curves derived for 5-Storey shear wall building based on Peak Ground Acceleration

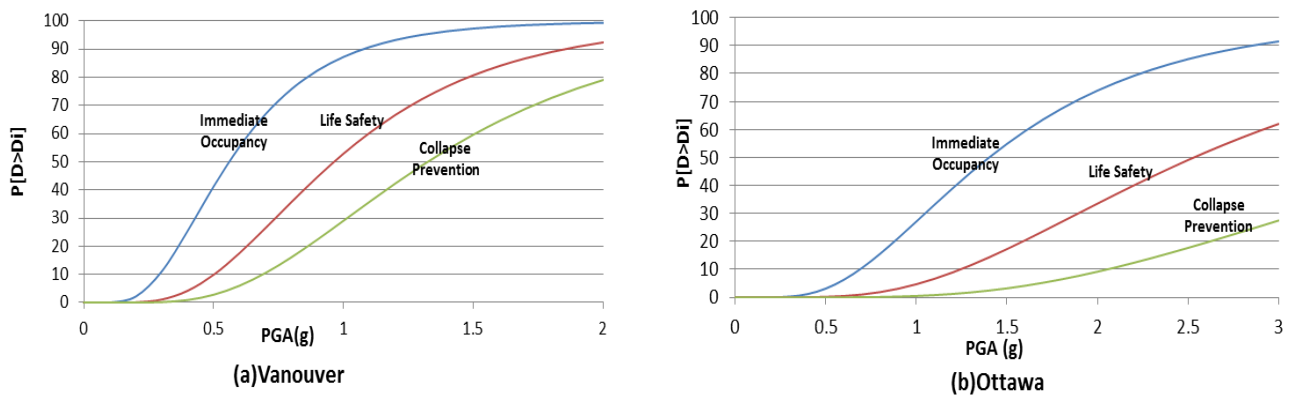


Figure 4-11: Fragility curves derived for 10-Storey shear wall building based on Peak Ground Acceleration

#### 4.4.3. Fragility curves as Affected by Year of Construction between 1975 and 2015

The fragility curves generated in the preceding section were developed for buildings designed on the basis of the 2010 NBCC and associated material standard CSA A23.3-2004. These buildings also represent buildings designed between 2005 and 2015, as the seismic design requirements in this period remained essentially the same with some changes in seismic hazard values in 2015. Examination of the evolution of seismic design provisions in Canada indicates that the same fragility curves may also apply to buildings designed after 1975, with some qualifications based on the seismic design

force levels. On the other hand, the ductility requirement affecting storey drift remained essentially the same between 1975 and 2015.

The 1975 NBCC (NRCC 1975) was the first edition of the code that introduced a base shear parameter ( $K$ ) to reflect the available ductility in seven different structural systems. This edition of the code made reference to the 1973 edition of CSA A23.3 (CSA 1973), which contained “special provisions for seismic design” for buildings designed to resist earthquake forces in a ductile manner. The provisions included general requirements and assumptions, as well as special design and detailing requirements for frame elements, connections, and walls. The Standard emphasized reinforcement detailing for concrete confinement, prevention of longitudinal reinforcement from buckling, minimum percentages for flexural reinforcement, longitudinal reinforcement detailing, joint shear design, prevention of brittle shear failure, and the sequence of energy dissipation among members. Therefore, 1975 was considered to be the benchmark year in Canada for significant changes in design methodology.

The ductile design requirements continued to improve since 1975. This is especially true for the 1985 NBCC, which made reference to CSA A23.3-84 (CSA 1984) with capacity design principles introduced to seismic design. However, the requirements of the 1975 NBCC played a pivotal role in defining the ductility limits used in the current investigation for shear walls. The trend in the variation of design base shear after 1975 is shown in Figure 4-12.

A new set of fragility curves was developed for buildings with significantly lower base shears to permit the interpolation of results for in-between cases. These include; the 2-storey building in Vancouver, designed based on the 1990-1995 NBCC (NRCC 1990, 1995) (Figure 4-13), and the 5-storey building in Ottawa, designed based on the 1985 NBCC (Figure 4-14). For the 2-storey and 10-storey buildings in Ottawa it was possible

to generate the fragility curves empirically by using the rate of change in storey drifts observed for the 5-storey Ottawa building. This is shown in Figure 4-15.



Figure 4-12: Design load level based on different versions of NBCC  
 C: Conventional, MD: Moderate Ductile, D: Ductile

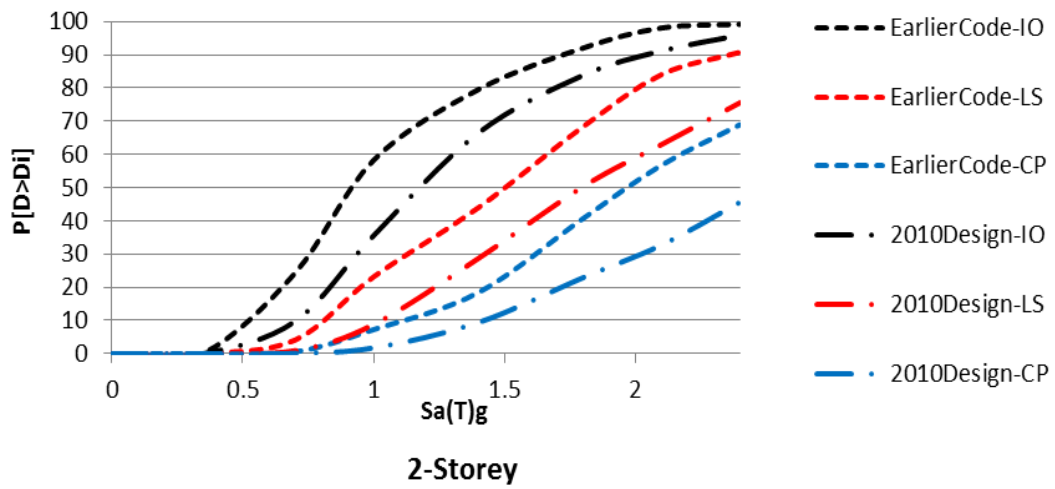


Figure 4-13: Fragility curves for 2-storey buildings designed based on 2010 NBCC and earlier codes in Vancouver

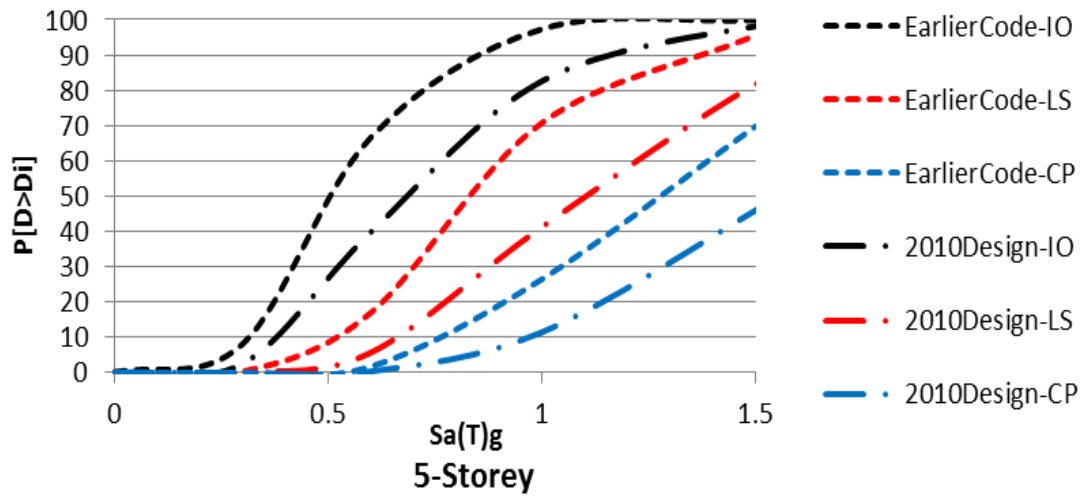


Figure 4-14: Fragility curves for 5-storey buildings designed based on 2010 NBCC and earlier codes in Ottawa

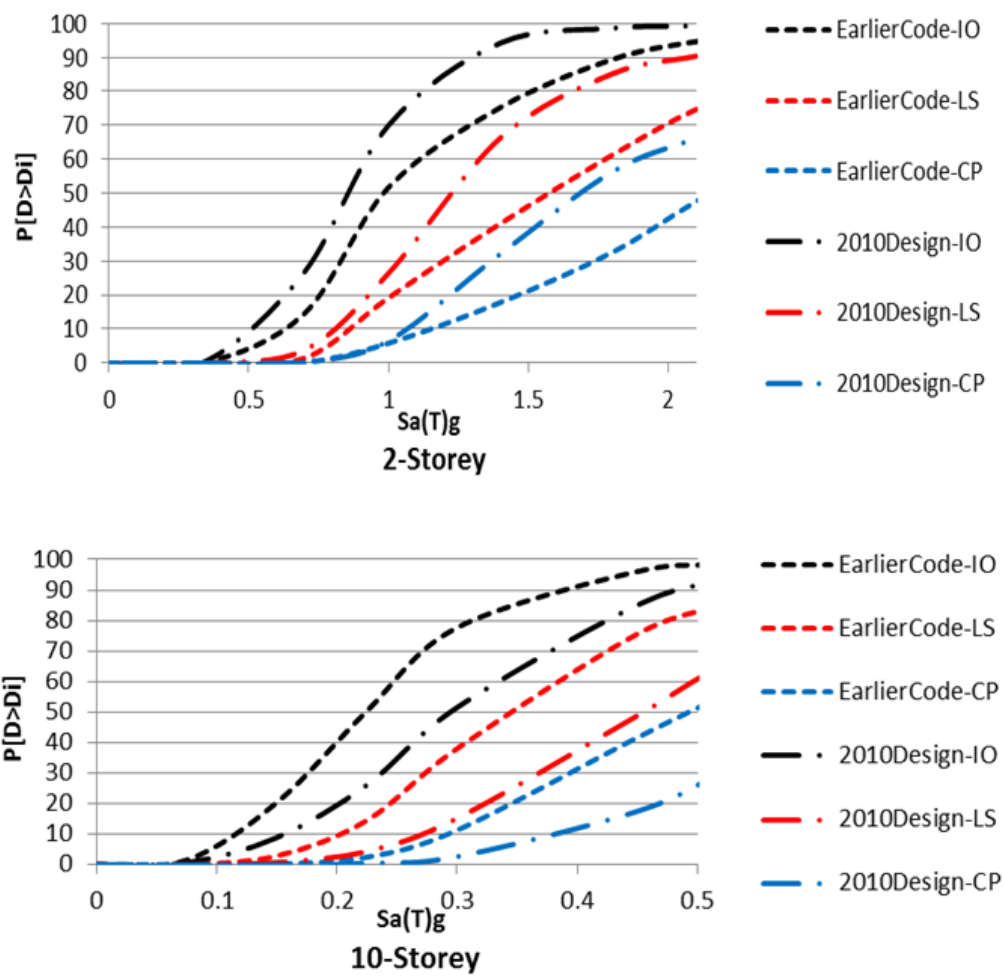


Figure 4-15: Fragility curves for 2- and 10-storey buildings designed based on 2010 NBCC and earlier codes in Ottawa

#### 4.5. Summary and Discussion

Due to the uncertainty and randomness inherent in seismic events, a probabilistic approach has been employed to assess the seismic vulnerability of reinforced concrete shear wall buildings. Fragility curves were developed to quantify the likelihood of structural damage as a function of ground motion intensity. A set of curves was generated first for 2-storey, 5-storey and 10-storey shear wall buildings in eastern and western Canada, designed on the basis of the 2010 NBCC. Another set was also developed for buildings designs that showed significantly different design force levels; or in the case of the 2-storey building in Vancouver, because of significant differences in ductility as discussed in the preceding section.

The fragility curves discussed above are used to demonstrate the seismic vulnerability of shear wall buildings selected in the current investigation. Two sets of assessment were considered for each region; those buildings that conform to the 2010 NBCC requirements, and those that were designed using an earlier edition of NBCC since 1975. Four different scenarios were examined for vulnerability assessment of buildings; i) 2010 UHS at the empirical code period, ii) 2010 UHS at maximum allowed code period (dynamic period with the limitation stated in NBCC), iii) amplified spectral intensity at the empirical code period, and iv) amplified spectral intensity at maximum allowed code period. The 2010 UHS was obtained from the 2010 NBCC for earthquake records having 2% probability of exceedance in 50 years. The amplified spectral intensity was obtained by using an amplification factor to the increase in UHS to 1% in 50 year earthquake based on  $S_a(T = 0.2)$ .

The percentage probabilities of exceedance for each performance level are tabulated in Table 4-3 and

Table 4-4. The results based on the empirical code periods indicate that 2-storey and 5-storey buildings recently designed in Vancouver met the life safety performance limit, exhibiting 5% and 10% probability of exceedance at the NBCC 2010 hazard levels. The 10-storey building designed for Vancouver has higher probability of life safety exceedance at the spectral acceleration corresponding to empirical code period, showing 50% probability of exceedance. For the same intensity level, the 2, 5 and 10-storey buildings showed 25%, 50% and 85% probability of exceeding the immediate occupancy performance limit, respectively. The 2-storey and 5-storey buildings in Ottawa indicated 0% probability of exceeding life safety, while the 10-storey building showed 3% of probability at code spectral acceleration corresponding to empirical period. The probabilities of exceeding immediate occupancy limit state for the same three buildings in Ottawa are 10%, 5% and 25%, respectively. For buildings designed based on the earlier editions of the code (dating back to 1975), the results are given in Table 4-3 and Table 4-4 for Vancouver and Ottawa for design codes that required the lowest design forces, and as expected they show higher probabilities of exceedance relative to 2010 NBCC designs.

The use of the fragility curves developed for buildings with different heights based on PGA as the intensity measure is illustrated in Figure 4-16 for the two locations considered. The vertical lines show code PGA values (representing seismic event with 2% probability in 50 years) and the amplified PGA with the same factor discussed above (representing seismic event with 1% probability in 50 years). The results show comparable seismic vulnerability values relative to those obtained from the curves based on spectral intensity. In summary, it may be concluded that the suite of fragility curves presented provide a powerful set of analytical tools for seismic vulnerability assessment of shear wall buildings in Canada built after 1975, since the implementation of ductile design practices.

Table 4-3: Probability of exceeding limit states at specific hazard levels for Vancouver

Building	Limit State	Vancouver 2010 Design				Vancouver Earlier Design			
		Code T		Maximum allowed T		Code T		Maximum allowed T	
		Code Spectrum	Amplified Spectrum	Code Spectrum	Amplified Spectrum	Code Spectrum	Amplified Spectrum	Code Spectrum	Amplified Spectrum
2-Storey	IO	25%	85%	10%	70%	55%	95%	20%	80%
	LS	5%	50%	1%	30%	20%	70%	5%	45%
	CP	0%	25%	0%	10%	8%	45%	1%	20%
5-Storey	IO	50%	92%	20%	80%	N/A	N/A	N/A	N/A
	LS	10%	60%	0%	35%				
	CP	3%	30%	0%	10%				
10-Storey	IO	85%	100%	40%	90%	N/A	N/A	N/A	N/A
	LS	50%	100%	5%	55%				
	CP	35%	90%	0%	30%				

Table 4-4: Probability of exceeding limit states at specific hazard levels for Ottawa

Building	Limit State	Ottawa 2010 Design				Ottawa Earlier Design			
		Code T		Maximum allowed T		Code T		Maximum allowed T	
		Code Spectrum	Amplified Spectrum	Code Spectrum	Amplified Spectrum	Code Spectrum	Amplified Spectrum	Code Spectrum	Amplified Spectrum
2-Storey	IO	10%	75%	0%	30%	25%	95%	0%	50%
	LS	0%	40%	0%	7%	2%	70%	0%	15%
	CP	0%	18%	0%	2%	0%	35%	0%	5%
5-Storey	IO	5%	50%	0%	27%	20%	85%	2%	50%
	LS	0%	15%	0%	2%	3%	50%	0%	10%
	CP	0%	2%	0%	0%	0%	15%	0%	1%
10-Storey	IO	25%	87%	2%	18%	45%	95%	5%	50%
	LS	3%	48%	0%	2%	10%	85%	0%	15%
	CP	0%	16%	0%	0%	1%	40%	0%	4%

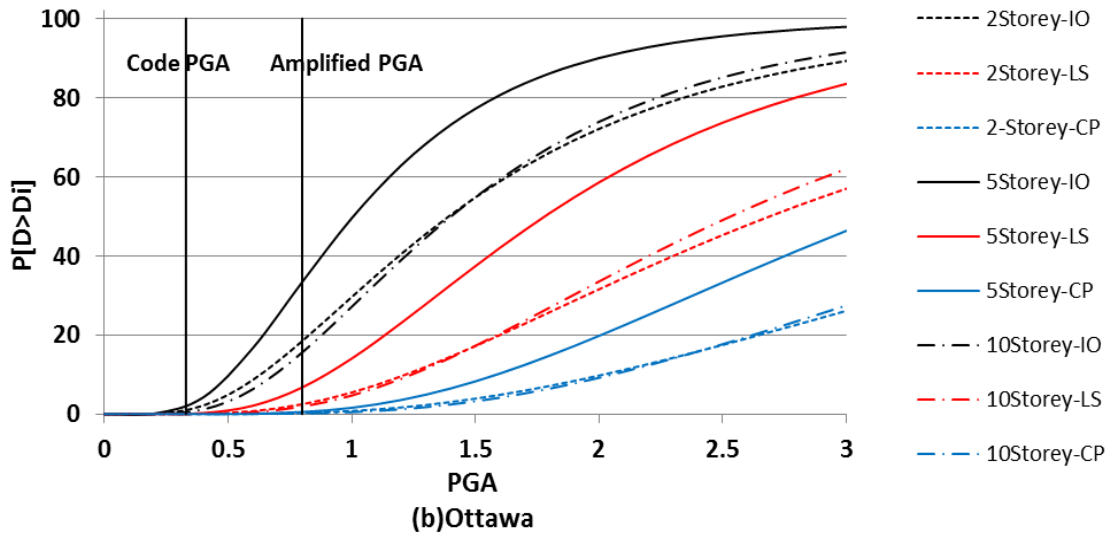
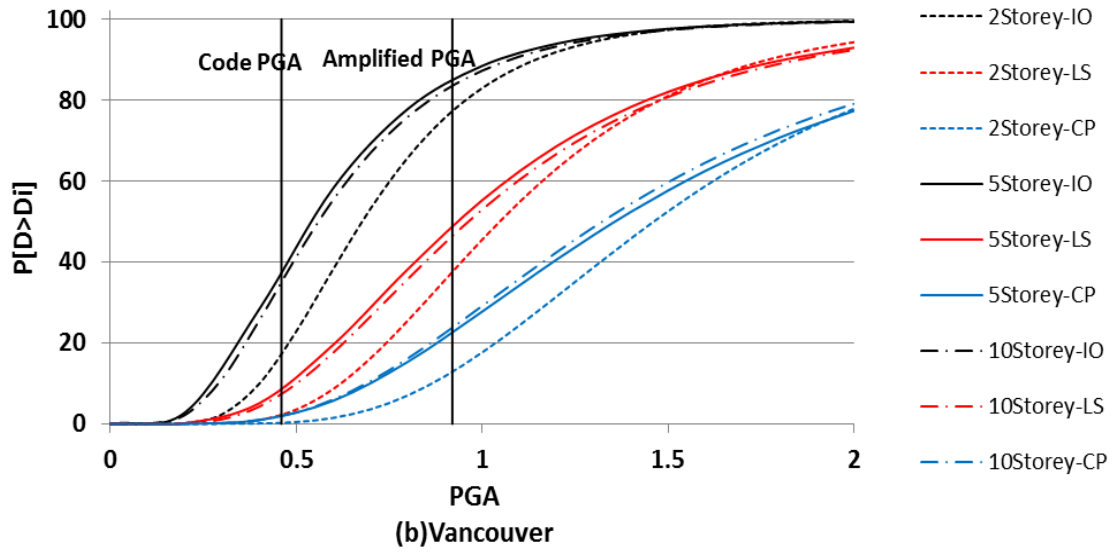


Figure 4-16: Fragility curves for 2010 buildings based on based on PGA compared to code PGA

# **Chapter 5. Seismic Fragility Analysis of Non-ductile Concrete Shear Wall Buildings in Canada**

**Abstract:** Seismic vulnerability of non-ductile reinforced concrete shear wall buildings in Canada, designed prior to 1975, is assessed. A probabilistic methodology was employed to generate fragility curves. Two, five and ten-storey buildings were considered for Vancouver and Ottawa, designed based on the 1965 and 1970 editions of the National Building Code of Canada, respectively, representing locations of high and medium seismicity in Canada.

Inelastic dynamic response history analyses were considered under site-specific earthquake records. Incremental Dynamic Analysis was employed for the development of seismic fragility curves. The results of fragility analyses are compared with those obtained earlier for buildings conforming to post-1975 seismic codes meeting ductile design requirements of modern seismic codes. It is concluded that the pre-1975 shear wall buildings in Vancouver have high seismic vulnerabilities. The buildings in Ottawa, designed and built in the same era perform reasonably well under the 2010 NBCC hazard

levels. The discrepancy in performance between the older and more recently design buildings increases as seismic intensity and building height increase.

## 5.1. Introduction

Seismic design provisions in modern building codes improved significantly in the mid-1970s with the incorporation of improved hazard values and ductile seismic design and detailing. Depending on the year of construction and the level of seismic design and detailing, a large number of existing reinforced concrete buildings is seismically vulnerable. Considering the probabilistic nature of seismic hazards, a probabilistic methodology is best suited for seismic vulnerability assessment of existing buildings.

A large inventory of shear wall buildings exists in Canada, many designed and constructed in the pre-1975 era. A country-wide seismic vulnerability assessment of these buildings necessitates their classification into groups with representative buildings selected in each group. Earlier codes were based on seismic design loads computed as a percentage of their weight, without the intricacies of modern seismic codes. The current investigation is intended to provide seismic vulnerability assessment of older reinforced concrete shear wall buildings systematically, as part of a comprehensive study that involves reinforced concrete and masonry buildings in Canada. Vulnerability assessment of post-1975 shear wall buildings are reported in Chapter 4.

Seismic vulnerability varies with site seismic hazard and adopted seismic design and detailing of buildings. Vancouver and Ottawa have been chosen in the current investigation as representative of western and eastern Canadian seismicity. British Columbia in western Canada is a province with the highest seismicity in the country with half of its population living in Great Vancouver area. In eastern Canada, Ottawa is a major metropolitan city located in a region of moderate seismicity. The results generated for Ottawa can be extended to cover Montreal, which is the largest city in eastern Canada

with similar seismic hazard values. Shear wall buildings having 2-, 5- and 10-storey heights, covering a range of building periods used in practice, have been selected as representative structures.

There has been limited research on seismic risk assessment of Canadian buildings. Ventura *et al.* (2005) studied seismic risk estimation in southwestern British Columbia, classifying buildings based on their material, lateral load bearing system, height, usage, and age to derive damage probability matrix and fragility curves for each building class in terms of Modified Mercalli Intensity (MMI). Other research for seismic vulnerability assessment of reinforced concrete (RC) buildings elsewhere in the World was limited to the study of RC frame buildings. Ramamoorthy *et al.* (2008) developed fragility curves for reinforced concrete frame buildings designed for gravity loads. Nonlinear pushover analyses were applied to formulate fragility estimation as a function of spectral acceleration using peak inter-storey drift as the response parameter. Celik and Ellingwood (2009) assessed seismic vulnerability of gravity load designed RC frames in Memphis, Tenn. with 2D modelling, using OpenSees and synthetic earthquake ground motions. Seismic fragilities were derived for low-rise, mid-rise, and high-rise gravity load designed RC frames based on the FEMA performance-based design objective. The results indicated that the majority of existing gravity load designed RC in Memphis are vulnerable for life safety and collapse prevention performance objectives. Masi and Vona (2012) evaluated seismic capacity of existing RC buildings designed for gravity loads only. They applied non-linear dynamic analysis on selected structural models for buildings with different ages, number of storeys, either with or without infill walls. Rajeev and Tesfamariam (2012a) developed seismic fragilities for non-ductile RC frames in 2D using OpenSees with fixed base model and models with soil-structure interaction. Bilgin (2013) studied seismic fragility of reinforced concrete public buildings with representative designs according to the 1975 version of the Turkish seismic design code applying 100 strong ground motion records with peak ground velocity as seismic intensity measure. Jeon *et al.* (Jeon et al. 2015) studied fragility

functions for non-ductile concrete frames using numerical simulation in Opensees by subjecting the models to ground motions representative of the seismic hazard in California. Ptilakis *et al.* (2014) assessed seismic vulnerability of RC frame buildings considering the effects of building age and soil-structure interaction. Karapetrou *et al.* (2015) also studied the effect of soil-structure interaction and site soil conditions on seismic performance of RC moment resisting frames. A 9-storey RC moment resisting frame, designed for low seismic effects, was chosen as the reference structure. Applying incremental dynamic analyses, fragility curves were derived as a function of peak ground acceleration for both fixed-base and soil structure interactive models (for linear elastic and nonlinear soil behaviours).

## 5.2. Selection and Design of Representative Buildings

Three shear wall buildings with 2-, 5-, and 10-storey heights and regular plan were selected as representative buildings having pre-1975 Canadian code designs. The floor plan consisted of five bays with 7.0 m span length in each direction and storey height of 4.0 m. Figure 5-1 shows the geometric details of the buildings.

In order to assess seismic vulnerability of buildings designed within a time period, covering a number of code cycles, one has to review the variation in seismic design force levels and the evolution of seismic design and detailing requirements in that time period. In Canada, seismic design force levels are defined in NBCC and design and detailing requirements are outlined in CSA A23.3. The design practice for shear wall buildings in Canada can be viewed in two categories: those performed prior to the enactment of ductile design principles (pre-1975 era), and those performed in the post 1975 era. The 1975 NBCC (NRCC 1975) made reference to CSA A23.3-1973 (CSA 1973), which was the first edition of the concrete design standard that implemented ductile design principles. The current paper provides seismic assessment of older existing buildings,

designed between 1953 and 1975, and hence a review of this era, as presented below, is on order.

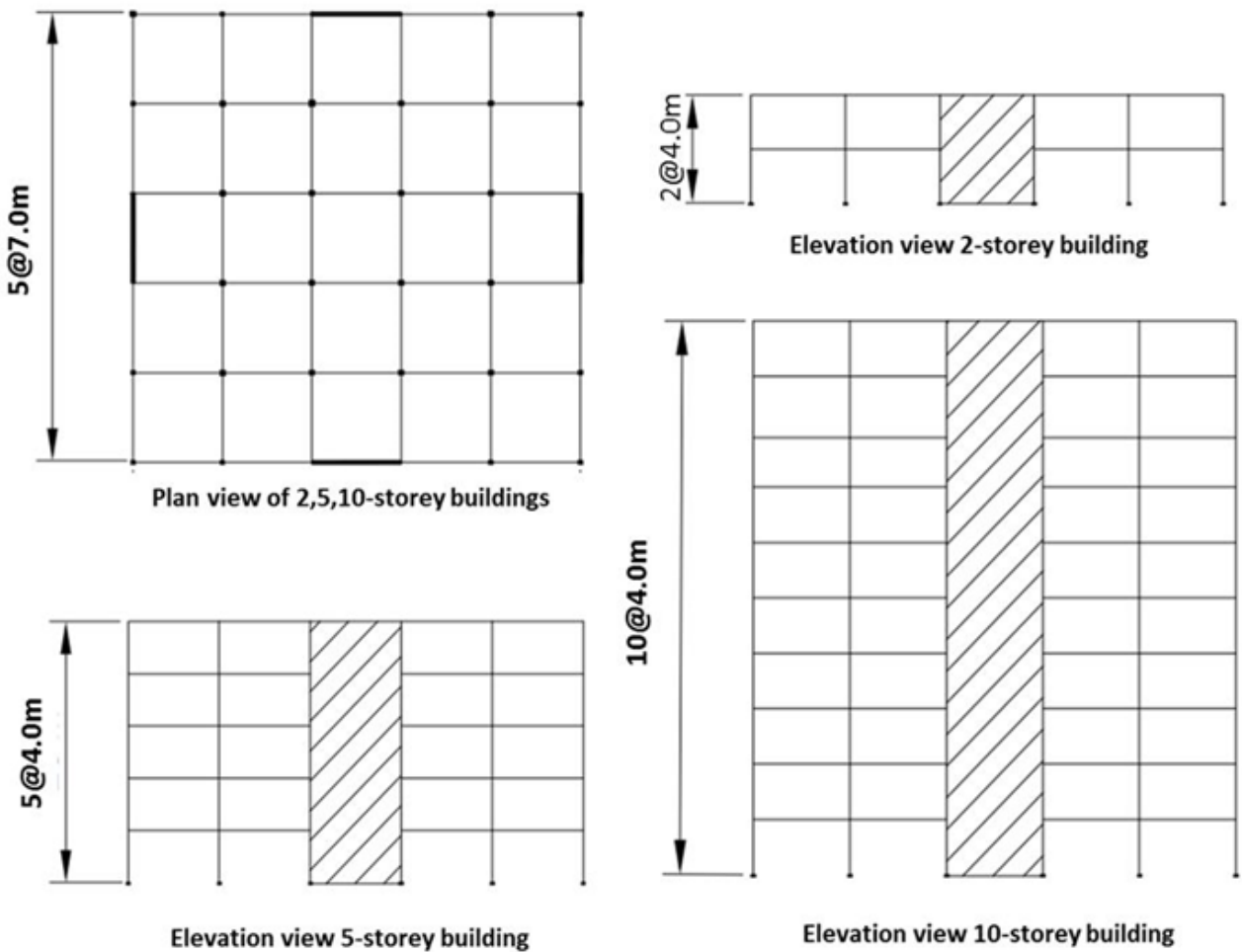


Figure 5-1: Geometry of selected buildings

Figure 5-2 shows the evolution of design seismic base shear over the years based on NBCC for the 2 -, 5-, and 10 storey shear wall buildings considered. The figure indicates that the 1970 gave the lowest seismic base shear for buildings in Ottawa and 1965 gave the lowest design forces for Vancouver though the variation among the years was small. Therefore, the selected shear wall buildings were designed conforming to the

requirement of the 1965 NBCC (NRCC 1965) for Vancouver, and the 1970 NBCC (NRCC 1970) for Ottawa.



Figure 5-2: Base shear factor of buildings with different height designed based on different codes in Ottawa and Vancouver (MD and D stand for moderately ductile and ductile, respectively)

All the buildings were designed to resist seismic lateral forces with two shear walls in each direction. The gravity loads included 1.9 kPa of live load and 1.0 kPa superimposed dead load in addition to the weight of the structural elements. The designs were carried out for 25 MPa concrete, representing the common concrete strength of older structures, and grade 400 MPa reinforcing steel. These buildings were designed without confined wall boundary elements. They did not contain buckling prevention ties or sufficient transverse reinforcement for diagonal tension, and hence they could develop shear failure before flexural failure.

### **5.3. Nonlinear Modelling and Analysis**

#### **5.3.1. Computer Software Selected and Non-linear Modelling of Shear Wall Buildings**

Computer software PERFORM 3D (CSI 2013), developed by Computers and Structures Inc., was selected as the analytical tool for nonlinear dynamic analysis. Three dimensional numerical models were generated for the buildings using their design and detailing properties. PERFORM 3D has a variety of options for nonlinear modelling of elements, with inelasticity introduced either through plastic hinges or material non-linearity assigned to fiber models. FEMA Beam and FEMA Column components in PERFORM 3D allow the user to divide each structural component into a hinge and an elastic segment, satisfying the relationship between end moments and end rotations. Strength loss for beams and columns are enabled, which is as an essential characteristic for the current research, as the level of applied incremental load can go beyond the capacity for some of the elements. FEMA beam and FEMA column components were implemented to simulate nonlinear beam and column elements assuming lumped plasticity defined in plastic hinges. The initial effective rigidities of  $0.35EI$  and  $0.70EI$  were assigned to beams and columns, respectively. Nonlinear flexural characteristics of these members were defined using moment capacity obtained from the sectional analysis conducted using software SAP2000 (CSI 2009). This formed the basis for the plastic hinges assigned to members at their ends. Hinge properties were based on the ASCE 41

(ASCE 2007) recommendations for members without ductile detailing. They consisted of initially linear behaviour, followed by post-yield region and strength degradation, which marked the beginning of member failure.

Response of shear wall buildings is primarily dominated by the behaviour of shear walls, which necessitates an accurate and detailed model for the walls. The shear wall model in PERFORM 3D was verified earlier against tests of large-scale shear walls, as well as an analytical model in OpenSees in Chapter 3, and was found to be representative of the intended behaviour. Accordingly, a fiber-discretized model was employed for the simulation of wall behaviour. This involved the integration of concrete and steel segments across the depth of the shear wall element. Hognestad's unconfined concrete (1951) model was used in the entire wall section as the pre-1975 walls did not have confined concrete. The analytical model also considered inelastic shear response. Perform 3D allows nonlinear shear behaviour in the form of stress-strain behaviour of the shear wall section. This relationship was assigned to the wall models to improve the accuracy of computed displacements in highly non-linear range of deformations. The non-linear shear model had a trilinear shear material behaviour with a descending branch. The slope of the stress-strain relationship is assumed to be equal to the shear modulus "G" until the shear strength of concrete is attained. The second branch was modelled with a linear segment having a slope equal to 6% of the initial slope. This segment was then followed by a straight line as the descending branch.

### **5.3.2. Incremental Dynamic Analysis**

Incremental dynamic analysis (Vamvatsikos and Cornell 2002b) is employed to estimate structural performance under different intensities of earthquakes. These analyses involve subjecting the structural model to a set of ground motion records, each scaled to multiple levels of intensity, resulting in curves of response parameter versus intensity level.

Artificial earthquake ground motions, generated by Atkinson (2004) for western and eastern Canadian seismicity, were selected first to conduct incremental dynamic analysis (IDA). These records are compatible with the uniform hazard spectra (UHS) specified for seismic design in the 2005 and 2010 National Building Code of Canada. The records were generated for earthquakes having 2% probability of exceedance in 50 years. In this study, the records generated for the reference soil Type C were used. A detailed discussion of the record selection is given in the Appendix A. A set of twenty records were selected for each site. Selected records were scaled to cover the entire range of structural response.

IDA involves selecting two parameters: seismic intensity measure and engineering demand parameter (damage indicator). Two different seismic intensity measures were considered consisting of; spectral acceleration of fundamental period with 5% of critical damping ( $Sa(T1),5\%$ ), and peak ground acceleration (PGA).  $Sa(T1)$  has been used in previous research studies including Vamvatsikos and Cornell (Vamvatsikos and Cornell 2002b) and Ellingwood *et al.* (Ellingwood, Celik, and Kinali 2007). This measure of intensity reflects both the characteristic of the earthquake and the structural period. It is defined in the National Building Code of Canada as a design parameter, and is frequently used by designers. On the other hand, scaling based on PGA gives a uniform format for the hazard, irrespective of the period of the structure, allowing comparison of the behaviour of buildings. Furthermore, scaling based on PGA is more straight forward and convenient to use. Inter-storey horizontal drift ratio was selected as the damage indicator. The use of inter-storey drift to define different limit states is quite common among engineers as it can be computed and rationalized easily.

The analytical models with the inelastic member properties discussed above were used to conduct inelastic dynamic response history analysis under the ground motion records selected. The use of a set of twenty records with different scale factors resulted in over 200 dynamic analyses for each model. The results are presented in the form of

IDA curves in Figure 5-3 for the pre-1975 buildings that were analyzed for western and eastern Canadian seismicity. The curves show the variation of the first storey drift with incrementally increasing earthquake intensity in terms of spectral acceleration values. The same results are also plotted on a log scale in Figure 5-4 using PGA as the earthquake intensity parameter. The IDA results are then used to develop seismic fragility curves. The generation of seismic fragility curves is presented in the following sections.

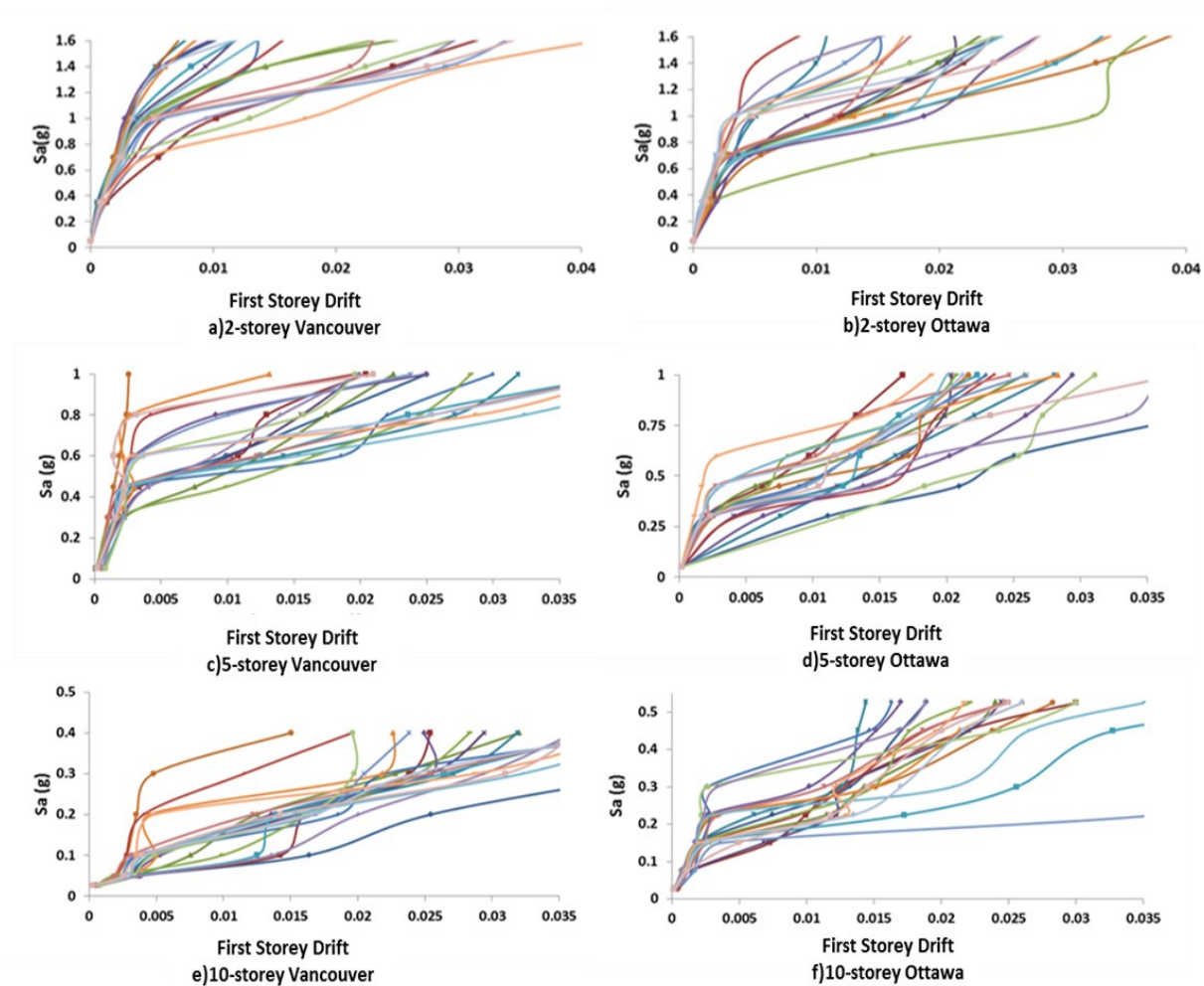
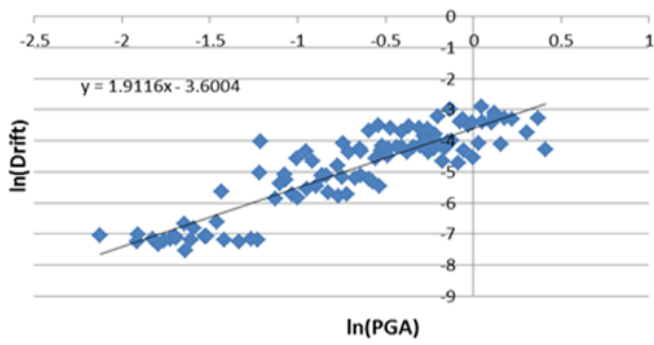
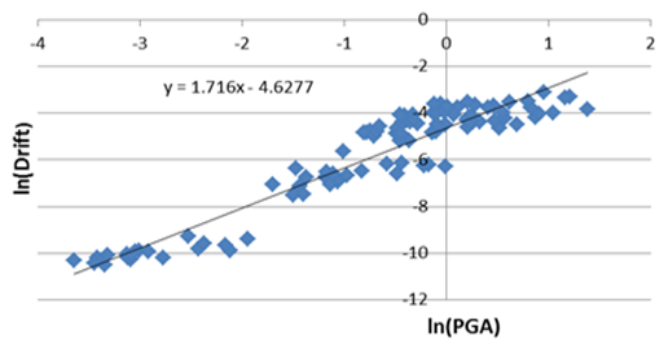


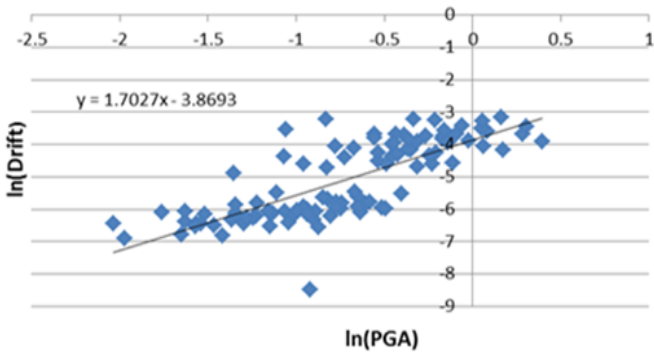
Figure 5-3: Result of IDA for pre-1975 shear wall buildings with 2-, 5- and 10-storeys heights



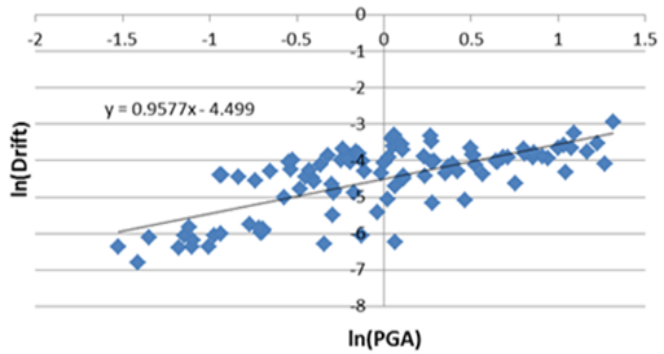
**a) 2-storey Vancouver**



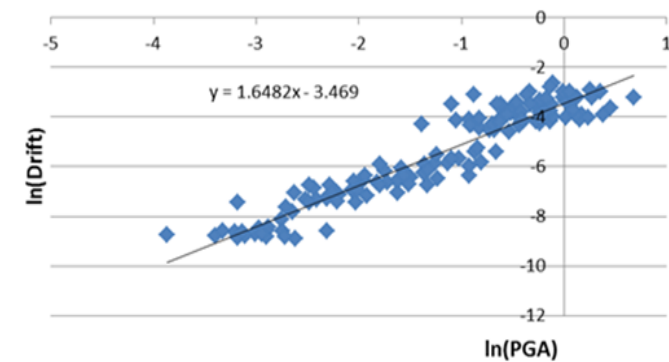
**b) 2-storey Ottawa**



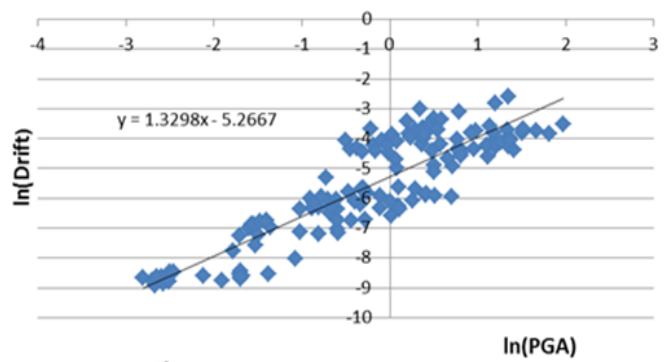
**c) 5-storey Vancouver**



**d) 5-storey Ottawa**



**e) 10-storey Vancouver**



**f) 10-storey Ottawa**

Figure 5-4: PGA-Drift relationship for pre-1975 shear wall buildings with 2-, 5- and 10-storeys heights

## 5.4. Development of Fragility Curves

### 5.4.1. Fragility Analysis and Seismic Risk Assessment

IDA results provide the input for developing fragility curves as probabilistic tools representing the probability of exceeding predefined damage states under different levels of ground motion intensity. The Fragility function is described in the form of Equation 5.1.

$$P[D > Di | IM] = \phi \left[ \frac{\ln(x / Di)}{\sqrt{\beta_{d/IM}^2 + \beta_c^2 + \beta_m^2}} \right] \quad \text{Eq. 5.1}$$

In the above expression,  $\phi(\cdot)$  is the standard normal cumulative distribution function,  $Di$  is the upper bound for each damage level,  $x$  is the median value of demand as a function of  $IM$ ,  $\beta_{d/IM}$  is the dispersion (logarithmic standard deviation) of demand conditioned on  $IM$ ,  $\beta_c$  is the capacity uncertainty and  $\beta_m$  is the modeling uncertainty.

The fragility curves are implemented in the risk assessment methodology to make a judgement on the performance of structures. Each level of damage can be representative of a certain level of loss in terms of money and downtime. Therefore, comprehensive risk assessment is a multi-disciplinary task that uses the output of seismic fragility analysis as the core ingredient.

The Pacific Earthquake Engineering Center (Cornell and Krawinkler 2000) gives an equation for the probabilistic seismic risk assessment framework. This framework suggests a relationship between a certain key Decision Variable,  $DV$ , such as annual seismic loss and a series of conditional probabilities to get mean annual frequency (MAF) of exceeding the decision variable,  $\lambda_{DV}$ , in terms of Damage Measure,  $DM$ , and ground motion Intensity Measure,  $IM$  given in Equation 5.2.

$$\lambda_{DV} = \iint G(DV | DM) dg(DM | IM) d\lambda_{IM} \quad \text{Eq. 5.2}$$

Where,  $G(DV | DM)$  is the probability that the decision variable exceeds specified values given that the engineering Damage Measures (herein, first storey drifts) are equal to particular values,  $G(DM | IM)$  is the probability that the Damage Measures exceed these values given that the Intensity Measure (herein, spectral acceleration at the fundamental period or PGA) is equal to particular values, and  $\lambda_{IM}$  is the MAF of the ground motion Intensity Measure.

The part  $G(DM | IM)$  in the equation is called Seismic Fragility Function. Seismic fragility relates the probability of reaching or exceeding predefined levels of damage to the severity of ground motion intensity. Analytical fragility curves are derived from the results of numerical simulations of a structure under artificial or historical earthquake records.

#### 5.4.2. Definition of Building Damage Limit States

IDA curves provide structural response. The assessment of structural damage requires limit states for different levels of damage that the buildings experience. Herein, three different limit states are used; i) Immediate Occupancy, ii) Life Safety and iii) Collapse Prevention. The description of these damage states as per ASCE 41 (ASCE 2007) is as follows:

- Immediate Occupancy: Building remains safe to be reoccupied. Lateral-force and gravity-load-resisting systems retain most of their design strengths.

- Life Safety: Building undergoes significant damage. Structural elements and components may be severely damaged, but gravity-load-carrying elements continue fulfilling their functions.
- Collapse Prevention: Substantial damage is imposed on structural elements. Significant strength and stiffness degradation of the lateral-load-resisting system is observed, and large permanent lateral deformations occur. The structure is not repairable and not safe to reoccupy.

The above definitions provide qualitative descriptions of damage states while quantitative definitions are required for the selected engineering demand parameters. The first-storey drift has been selected as a damage parameter in the current research project. The ASCE 41 recommendation of 0.5% drift ratio for shear wall buildings was selected as the threshold for immediate occupancy. Observations made from the IDA curves confirm that until this drift value is exceeded the structural behaviour remains essentially linear, allowing the building to be immediately re-occupied. ASCE 41 suggests a value of 1.5% drift ratio for the collapse prevention limit state of shear wall buildings with possibility of shear failure. The threshold for life safety limit state is assumed to be 1% drift as recommended by ASCE 41 for shear wall buildings. This provides a good estimate for life safety performance as it is in the middle of the other two limit states.

Federal Emergency Management Agency provides another platform for assessment of existing structures. It provides empirical fragility curves based on experts' opinion by dividing the design level into high code, moderate code, low code and pre-code in HAZUS MR4 manual (FEMA 2003). Damage state defined in this document are referred as minor (I), moderate (II), extensive (III) and complete collapse (IV). It also categorizes building design to pre-code, low-code, medium-code and high-code. The buildings designed between 1953 and 1975 in Canada fall in the low-code definition. HAZUS

values of maximum inter-storey drift suggested for low code shear wall buildings are presented in Table 5-1.

**Table 5-1: Inter-storey drift Limits for each damage state (FEMA 2003)**

<b>Building Type</b>	<b>Limit States - % Inter-storey Drift</b>			
	<b>Slight</b>	<b>Moderate</b>	<b>Extensive</b>	<b>Complete</b>
<b>Low-rise Shear Wall</b>	0.40	0.76	1.97	5.00
<b>Mid-rise Shear Wall</b>	0.27	0.51	1.32	3.33
<b>High-rise Shear Wall</b>	0.20	0.38	1.00	2.50

#### 5.4.3. Results of Seismic Fragility Analysis

Figure 5-5 shows the results of fragility analyses for 2-, 5- and 10-storey shear wall buildings designed prior to 1975 for the ASCE limit states. The figure also includes fragility curves developed earlier by the authors for the same shear wall buildings designed on the basis of the 2010 NBCC (NRCC 2010) requirements, also based on the ASCE limit states. The latter curves provide a means for comparison between the seismic vulnerabilities of older and newer buildings in Canada.

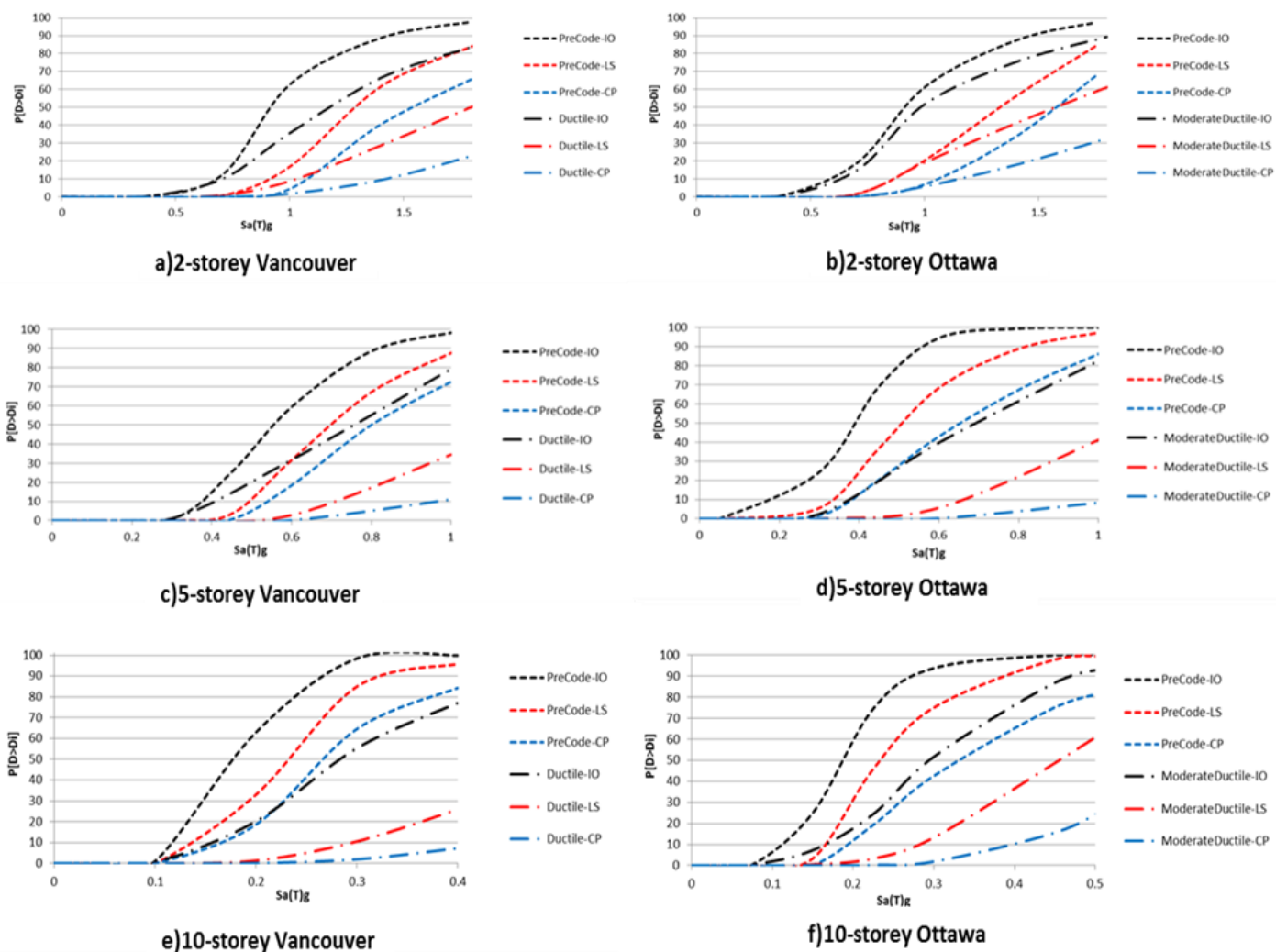


Figure 5-5: Comparative seismic assessment of 2-, 5- and 10-storey shear walls designed in different periods

Another set of fragility curves were generated using HAZUS limit states presented in Table 5-1 to provide a verification of the fragility curves against another commonly used limit states. The HAZUS values vary with building height (mid-rise and high-rise definitions in this document are compatible with the 5-storey and 10-storey cases in this study). The values for mid-rise and high-rise limit states were adopted for the 1965 NBCC code buildings designed for Vancouver, intended to cover pre-1975 building

designs. The suggested limit state values for low-rise buildings in HAZUS were far from the damage observed for the 2-storey building analyzed in the current study. Table 5-2 gives median PGA values of fragility curves for each damage state in the HAZUS document based on expert opinions. Figure 5-6 and Figure 5-7 show fragility analysis results based on HAZUS limits for 5-storey and 10-storey building in Vancouver. The PGA value corresponding to 50% probability of exceeding a given limit state can be compared with the corresponding value in Table 5-2, which is based on HAZUS. The values derived for the 10-storey building match the values in the Table, while the values for the 5-storey building in the Table are more conservative than the actual analytical results.

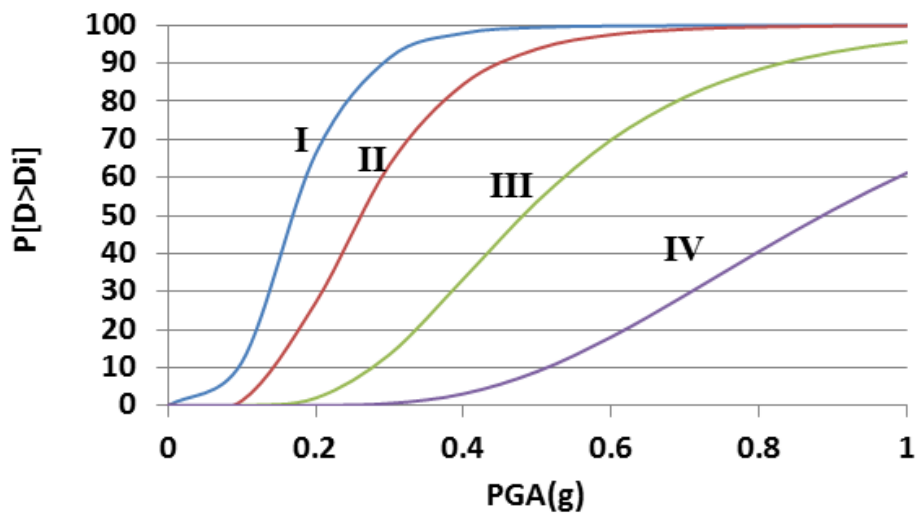


Figure 5-6: Fragility curves of pre-1975 5-storey shear wall in Vancouver

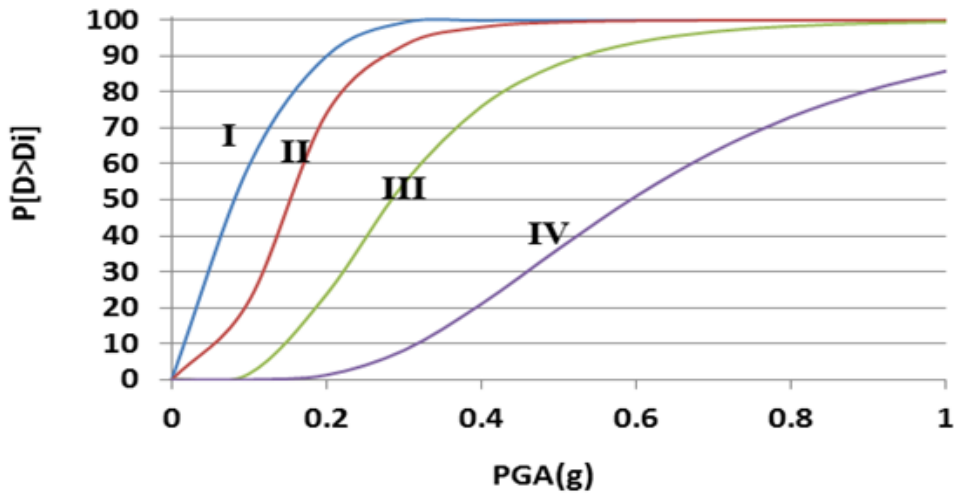


Figure 5-7: Fragility curves of pre-1975 10-storey shear walls in Vancouver

Table 5-2: Median anticipated PGA of fragility curves for each damage state (FEMA 2003)

Building Type	Expected Median PGA			
	Slight	Moderate	Extensive	Complete
Mid-rise Shear Wall	0.12	0.17	0.32	0.54
High-rise Shear Wall	0.09	0.15	0.27	0.44

#### 5.4.4. Effect of the Selection of Site Specific Records

The fragility curves presented earlier were generated using eastern and western seismicity for Ottawa and Vancouver. Hence, the seismicity changed with building design. To investigate the effect of earthquake frequency content on fragility analysis the Ottawa buildings were analyzed also under western Canadian records. The results are compared in Figure 5-8 with those generated based on the eastern records, and indicate very little impact of using western record for the 2-storey building. The effect is more pronounced for 5- and 10-storey buildings. In these buildings, the western seismicity becomes more damaging at higher spectral accelerations when life safety and collapse prevention limit states are considered.

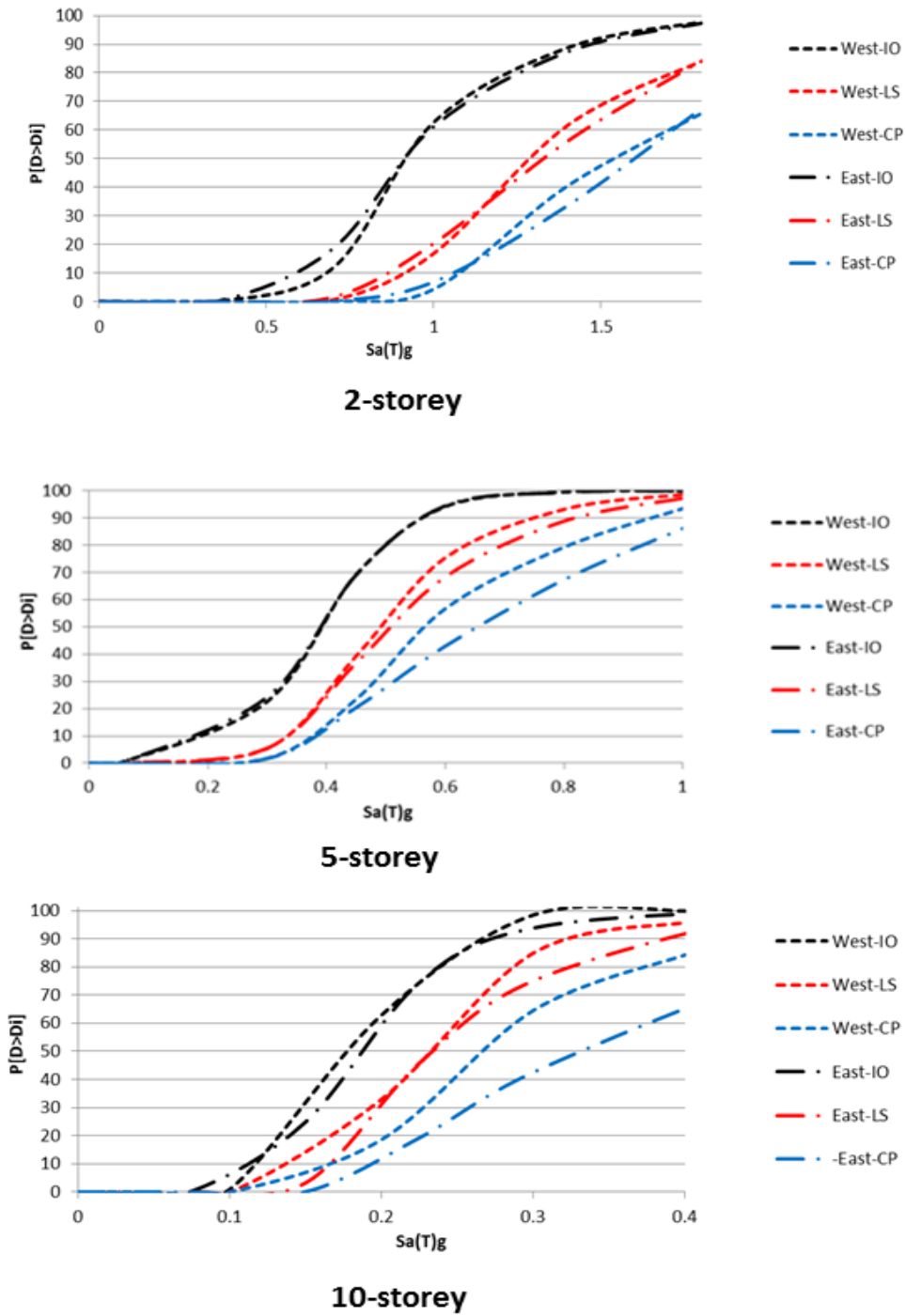


Figure 5-8: Shear wall buildings designed for Ottawa under western and eastern seismicity

### 5.4.5. Comparative Assessment of Shear Wall Buildings with Different Heights

The effect of building height on seismic vulnerability of buildings can be best assessed either by comparing the fragility curves based on PGA (as they are not period dependent) or by comparing fragility curves based on normalized spectral accelerations. The normalization was done by dividing spectral accelerations by the design spectral acceleration for 2% in 50 year hazard with a return period of 2475 years. The comparisons are shown in Figure 5-9 and Figure 5-10, and indicate distinct differences in seismic hazard based on building height. While the comparisons based on PGA do not show a pattern in variation with height, those based on normalized spectral accelerations show progressively decreasing damage from 10-storeys to 5 and 2-storeys in building height for all target performance limits.

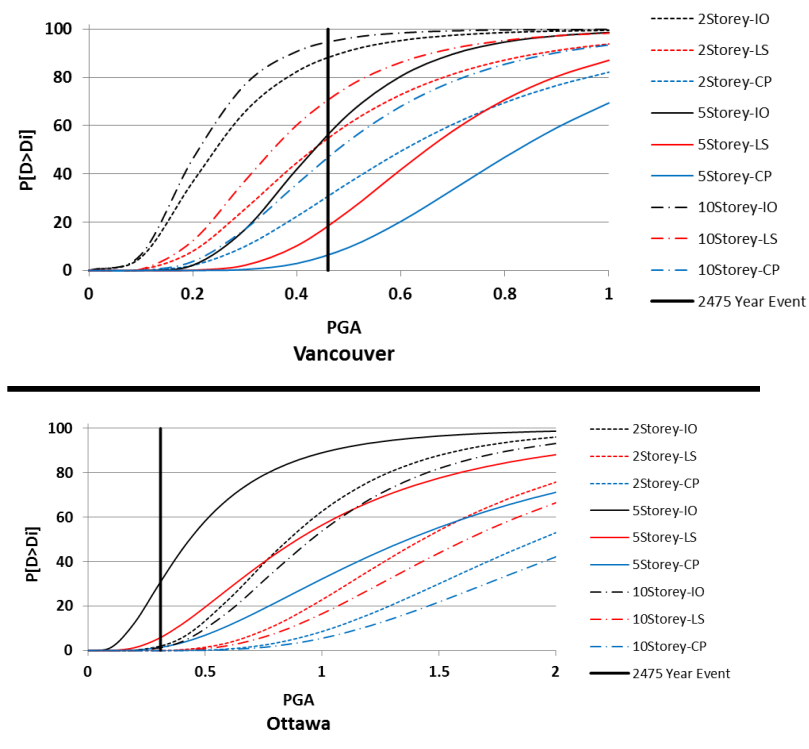


Figure 5-9: Comparison of behaviour of pre-1975 buildings with different heights based on PGA

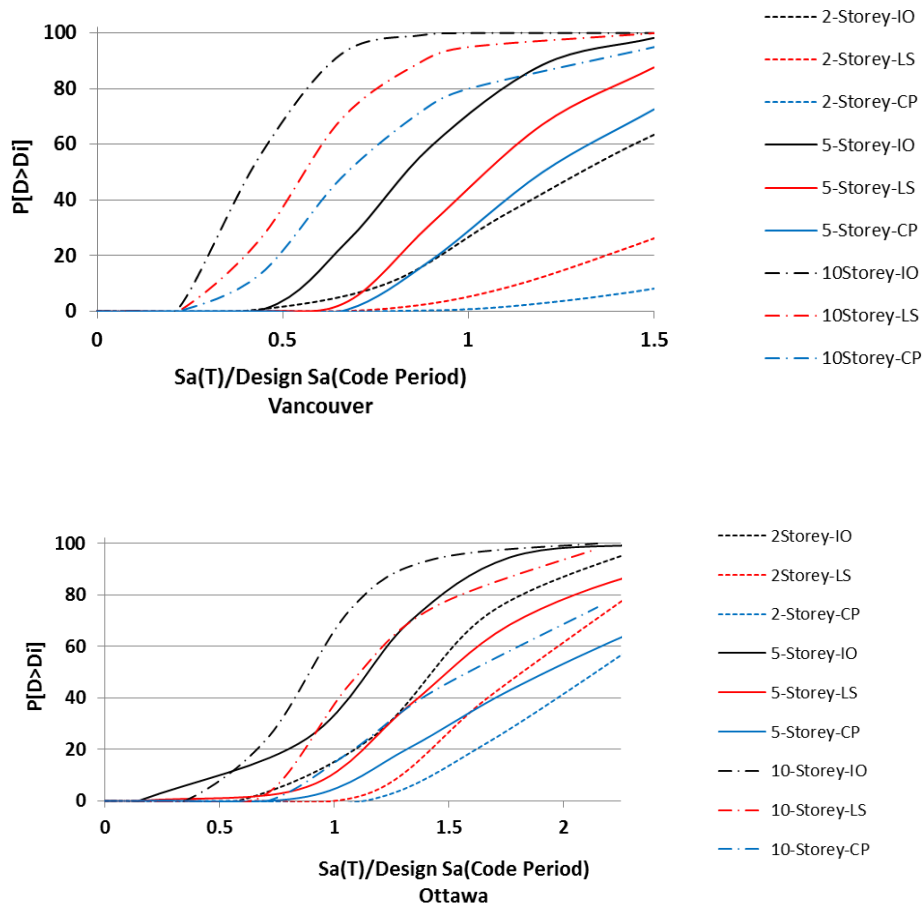


Figure 5-10: Comparison of behaviour of pre-1975 buildings with different heights based on Sa

## 5.5. Summary and Discussion

A probabilistic approach was implemented to generate analytical tools for seismic vulnerability assessment of shear wall buildings in the form of fragility curves. This was found appropriate because of the uncertainty and randomness inherent in seismic events. The curves help quantify the likelihood of structural damage as a function of ground motion intensity. They were developed for reinforced concrete shear wall buildings designed in the pre-1975 era in Vancouver and Ottawa. PERFORM3D software was used to generate nonlinear response quantities using two sets of twenty synthetic records for western and eastern Canadian

seismicity. Incremental Dynamic Analysis procedure was followed to cover the entire range of structural behaviour. Spectral acceleration at the fundamental period of the structure, and peak ground acceleration were chosen as the ground motion intensity measures. The first storey drift was used as the damage indicator. Table 5-3 summarizes probabilities of exceeding different damage limit states for the 2- 5- and 10- storey shear wall buildings considered. The probabilities of exceedances were assessed at spectral accelerations based on 2% in 50 year hazard values. They corresponded to the periods computed either by the empirical expressions given in the 2010 NBCC, or by using dynamic periods as permitted by the code. The results indicate that pre-1975 shear wall buildings in Canada show high vulnerabilities under the 2010 NBCC hazard values, especially if the empirical period expressions are used. Taller shear wall buildings show more vulnerability relative to mid-rise and low-rise buildings. Buildings in Vancouver are significantly more vulnerable than those in Ottawa.

**Table 5-3: Summary of fragility probabilities for buildings with pre-modern seismic code design**

Building	Limit State	Vancouver 1965		Ottawa 1970	
		Code T	Maximum allowed T	Code T	Maximum allowed T
2-Storey	IO	60%	10%	10%	5%
	LS	15%	0%	0%	0%
	CP	2%	0%	0%	0%
5-Storey	IO	75%	35%	30%	25%
	LS	45%	10%	10%	5%
	CP	30%	5%	5%	2%
10-Storey	IO	100%	90%	65%	10%
	LS	100%	70%	35%	0%
	CP	90%	50%	15%	0%

# Chapter 6. Seismic Performance Assessment of Shear Wall Buildings with Plan and Elevation Irregularities

**Abstract:** Structural irregularities result in increased seismic force and deformation demands, increasing seismic vulnerabilities, especially if their effects are not accounted for in design. Seismic fragility analysis of non-ductile shear wall buildings with plan and elevation irregularities was conducted, and fragility curves were developed for pre-1975 shear-wall buildings in Canada. Three building heights were considered and 2- 5- and 10-storey buildings were designed for Vancouver in western Canada as a representative city in a high seismic region, and for Ottawa in eastern Canada as a city in a medium seismic region. Analytical models were developed for the buildings for non-linear response history analysis. Incremental dynamic analysis was employed to generate probabilistic fragility data. The results were presented in the form of fragility curves for each building in each region. Spectral and peak ground accelerations were used as seismic intensity parameters, and inter-story drift was used for as the seismic demand parameter. The results show that building irregularities increase seismic vulnerability of non-ductile shear wall buildings. The level of increase in the probability of exceedance of pre-

established performance limits depends on the type and degree of irregularity, as well as on the intensity of seismic hazard as demonstrated in the paper.

## 6.1. Introduction

Fragility curves provide probabilistic tools for pre-earthquake disaster planning and seismic retrofitting. Seismic fragility curves can be developed based on expert opinion, damage performance during past earthquakes or through analysis of representative buildings. Qualitative approaches for fragility analysis become more difficult for irregular buildings. Therefore analytically-derived fragility relationships provide an attractive alternative for seismic vulnerability assessment of irregular buildings. Countless possibilities of the type and level of irregularities that exist in practice provide challenges, making it difficult to categorize irregular buildings for the purpose of fragility curve development. Geometric irregularities in both plan and elevation can be quantified based on current codes and standards (NRCC 2010, ASCE 2014) for selected building layouts, covering the two common irregularity types that have led to the majority of building damages in past earthquakes. The behaviour of buildings having these types of irregularities has been investigated in the current research project.

Previous research in the area is scarce. A few researchers have addressed seismic assessment of reinforced concrete buildings with irregularities, as presented below.

**Plan Irregularity:** Jeong and Elnashai (2007) developed an analytical methodology for the derivation of fragility functions for structures with plan irregularities, employing Park and Ang spatial (3D) damage index. The authors suggested a new spatial damage measure in fragility analyses of structures with significant torsional and bi-directional effects. Gokdemir *et al.* (Gokdemir et al. 2013) assessed the effects of torsional irregularity on buildings with different number of storeys, using SAP2000, and concluded that separating building sections from each other with proper separation distances and increasing lateral rigidity in the weak direction of structures reduces the effect of torsional irregularity.

**Elevation Irregularity:** Athanassiadou (2008) studied multi storey reinforced concrete (R/C) frame buildings with irregularities in elevation. Two 10-storey two-dimensional plane frames with two and four large setbacks in upper floors, and another building which was regular in elevation, all designed based on the provisions for high and medium ductility of the 2004 Eurocode8 (Eurocode 2005), were considered. SAP2000 generated models were subjected to inelastic static pushover analysis and inelastic dynamic time-history analysis for assessment of their seismic performance using global and local criteria. Rajeev and Tesfamariam (Rajeev and 2012b) assessed fragility of 3, 5, and 9-storey RC gravity load frame buildings, designed prior to 1970s, using soft storeys and the quality of construction as parameters. The authors used the response surface method to develop a predictive equation for seismic demand as a function of soft storey and construction quality. They concluded that the results were sensitive to the interaction of soft storey and construction quality. Jeong *et al.* (2012) conducted fragility analyses of twelve mid-rise buildings. Eurocode-compliant reinforced concrete buildings with different heights, ductility levels, and design peak ground acceleration were considered with regular and irregular layouts. The authors applied Incremental dynamic analyses using ZEUS software in 2D. Based on the observation of damage state probabilities of wall-frame structures designed to high PGA and ductility levels, these buildings did not satisfactorily achieve favorable safety objectives. The researchers further concluded that buildings other than the wall-frame building considered satisfied the life safety performance level at design intensity levels. Moment resisting frame buildings with elevation irregularities designed by a modern seismic code demonstrated acceptable seismic performance. This implied that appropriate structural compensations have been implemented in their design to overcome unfavorable irregularity conditions.

## 6.2. Buildings Classification Based on Irregularities

Recent seismic codes impose restrictions on designing buildings with irregularities. Older seismic codes did not have such limitations. Therefore, there is a large number of

existing buildings for which the presence of irregularities has not been addressed at the design stage. The presence of irregularities in buildings affects both capacity and demand. Elevation irregularities can cause non-uniform distribution of damage along the building height, and result in storey failures, while plan irregularities can cause non-uniform damage distribution in plan.

The National building code of Canada (NRCC 2010) and ASCE41 (ASCE 2007, 2013) have been used in the current research for definitions and procedures related to seismic performance of irregular buildings. Division B in Part 4 of NBCC 2010 defines eight different types of irregularities. The presence of one or more of these irregularities triggers the requirement for dynamic analysis in design. ASCE 41 defines six different irregularity types. Five types of these irregularity types are common to both documents with either the same or different definitions, as listed in Table 6-1.

#### **6.2.1. Irregularities Investigated in This Study**

Regular and irregular shear wall buildings designed based on 1965 NBCC (NRCC 1965) for Vancouver were selected as representative shear wall buildings in Canada designed prior to 1975. This edition of the NBCC provides the lowest design base shear for buildings designed and constructed between 1953 and 1975. The geometry of regular 2-, 5-, and 10-storey buildings is presented in Figure 6-1. Figure 6-2 illustrates the plan view of building cases with plan irregularities compared to the regular plan view. The geometry of 5-storey buildings with elevation irregularities is demonstrated as sample in Figure 6-3, with similar change in the elevation for 2- and 10-storey cases. The floor plan consisted of five bays with 7.0 m span length in each direction and storey height of 4.0 m. All the regular buildings were designed to resist seismic lateral forces with two shear walls in each direction. The gravity loads included 1.9 kPa of live load and 1.0 kPa super-imposed dead load in addition to the weight of the structural elements. The designs were carried out for 25 MPa concrete, representing the common concrete strength of

older structures, and grade 400 MPa reinforcing steel. These buildings were designed without confined wall boundary elements. They did not contain buckling prevention ties or sufficient transverse reinforcement for diagonal tension, and hence they could develop shear failure before flexural failure.

Irregularities studied herein are divided into two main categories; irregularities in plan and irregularities in elevation. The elevation irregularity cases studied are shown in Figure 6-3. These cases comply with the vertical stiffness irregularity definition of NBCC (soft storey definition in ASCE), and the vertical geometric irregularity definition in both documents.

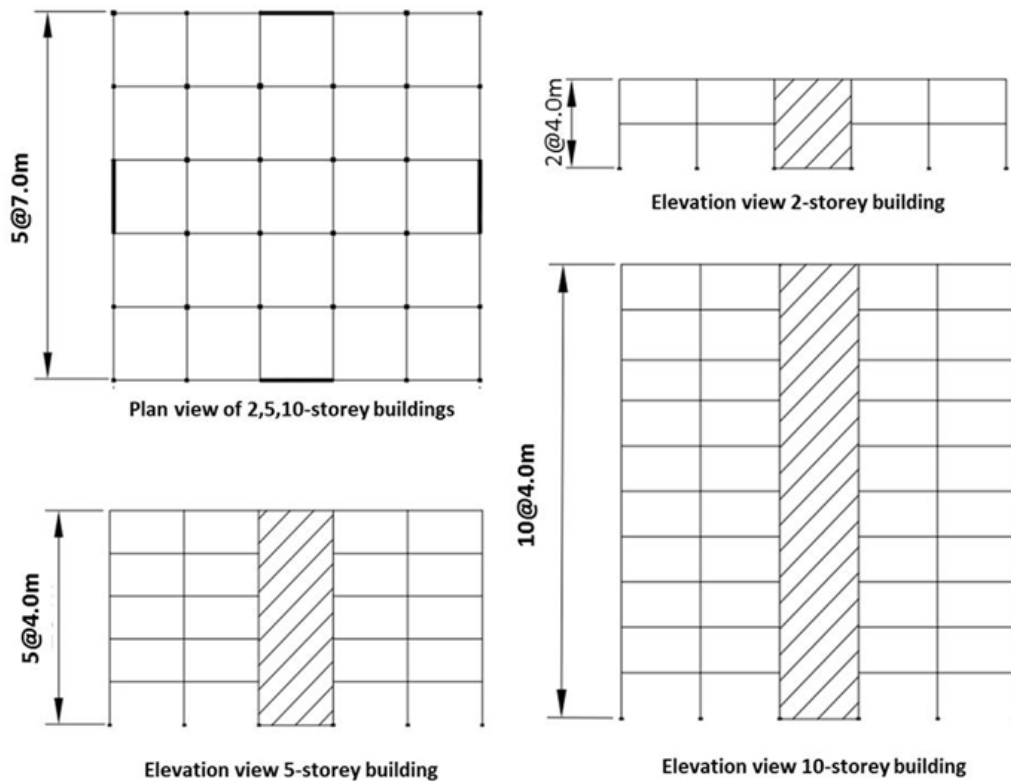


Figure 6-1: Geometry of regular buildings

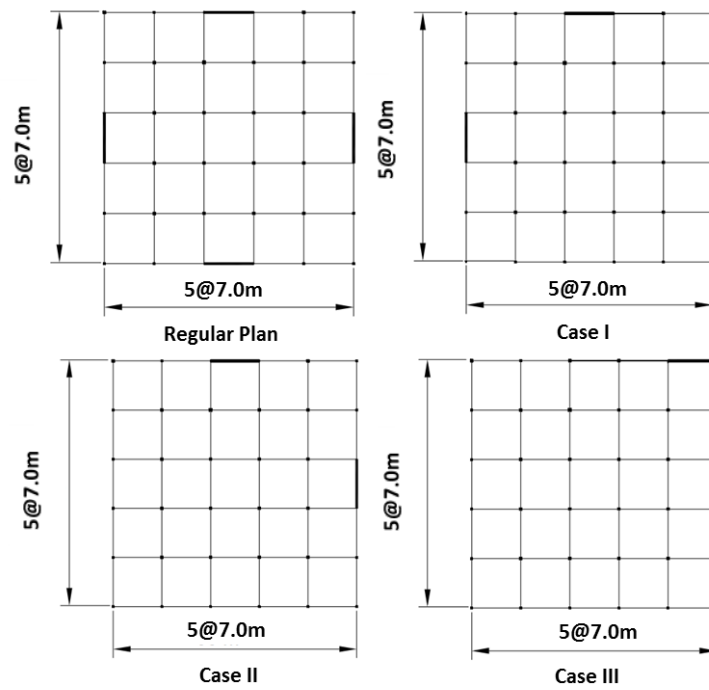


Figure 6-2: Plan view of regular buildings and the buildings with torsional sensitivity studied

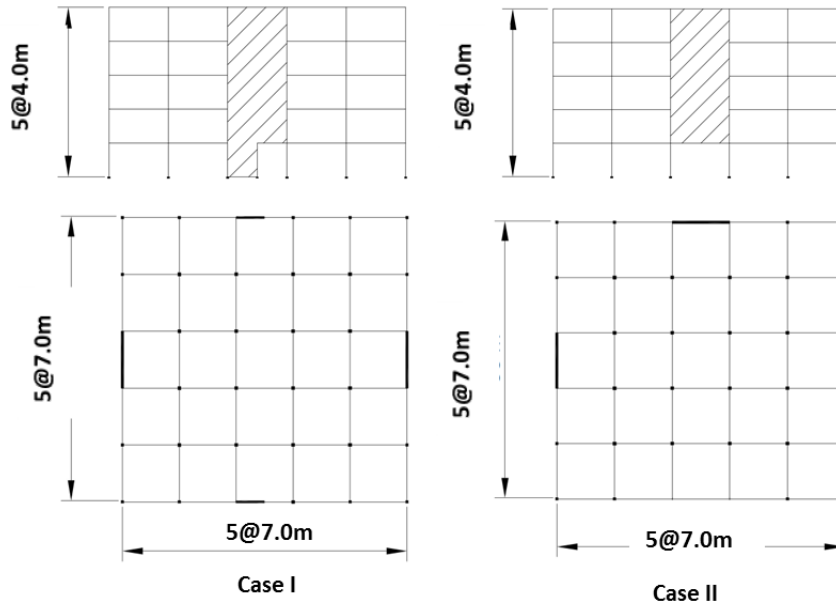


Figure 6-3: Plan and elevation view of irregular buildings with soft storey

The plan irregularities studied are referred to as torsional sensitivity in both documents. The buildings were designed using the effects of irregularities outlined in the 1965 NBCC. It is not practical to study all possible irregularities as the number of cases can be prohibitive. Therefore, possible extreme cases were studied for both plan and elevation irregularities. Intermediate cases are left to the judgement of the reader.

### 6.2.2. Plan Irregularity Cases Studied

According to the NBCC2010, torsional sensitivity shall be considered to exist when diaphragms are not flexible and the ratio B defined in NBCC 4.1.8.11(9) exceeds 1.7 in any direction. This ratio is defined as  $B_x = \delta_{\max} / \delta_{\text{avg}}$ , in which,  $\delta_{\max}$  is the maximum storey displacement at extreme points of the floor at level x in the direction of the earthquake force induced by the equivalent static forces acting at distances  $\pm 0.01D_{nx}$  from the centers of mass at each floor and  $\delta_{\text{avg}}$  is the average of displacements at extreme points of the structure at level x produced by the mentioned force.

Removing one shear wall in one direction throughout the building height would result in torsional sensitivity of the model. The effect would be substantially aggravated if one wall in each direction is moved, even though applied load is parallel to one axis. Applying the ASCE definition of torsional irregularity to Case I and Case II, the distance between the center of mass and center of rigidity becomes 50% of the plan dimension perpendicular to the direction of loading in one direction for the former case and in two directions for the latter case. When the NBCC definition of torsional sensitivity is used, the value of B factor based on the equivalent static loads becomes 1.8 and 2.0 for case I and case II, respectively. Case II and Case III are both extreme cases, having the highest possible value for the B parameter, which is equal to 2.0. Modal periods of regular buildings are listed in Table 6-2. Table 6-3 provides modal periods for buildings with plan irregularities.

**Table 6-1: Comparison of Definition of Irregularities Existing in both NBCC and ASCE**

<b>NBCC2010 Definition</b>	<b>ASCE Definition</b>
<b>Vertical stiffness irregularity</b>	
Vertical stiffness irregularity shall be considered to exist when the lateral stiffness of the SFRS in any storey is less than 70% of the stiffness in any adjacent storey or less than 80% of the average stiffness of the three storeys above or below.	<b>Soft Storey</b> Similar Definition to NBCC
<b>Weight (mass) irregularity</b>	
Weight irregularity shall be considered to exist where the weight of any storey is more than 150% of the weight of adjacent storey, not considering the roof.	<b>Mass Irregularity</b> Similar Definition to NBCC
<b>Discontinuity in capacity-Weak storey</b>	
A weak storey is the case when the shear strength is less than that in the storey above. The storey shear strength is the total strength of all SFRS elements for the direction under consideration.	<b>Weak storey</b> Shear strengths of the seismic-force-resisting system in any story in each direction is less than 80% of the strength in the adjacent story above.
<b>Torsional sensitivity</b>	
Torsional sensitivity shall be considered to exist when diaphragms are not flexible and the ratio B defined in NBCC 4.1.8.11(9) exceeds 1.7 in any direction: $B = \delta_{\max} / \delta_{\text{avg}}$	<b>Torsion</b> Torsional irregularity exist when the estimated distance between the story center of mass and the story center of rigidity is higher than 20% of the building width in either plan dimension.

### 6.2.3. Elevation Irregularity Cases Studied

The first elevation irregularity case studied produced a soft first storey by reducing the wall length for both walls in the x direction. This condition formed Case I, and did not create torsional sensitivity as the building remained symmetric. The second case (Case II) involved the removal of one of the walls at the first-storey level, in the direction of analysis, leaving the first storey not only as a soft storey but also as a torsionally sensitive floor. Storey drift was used as the damage indicator for both Case I and Case II, though the storey drift was the highest at the exterior most frame columns where the wall was removed, causing the highest lateral sway in the frame. This implies that in Case II the highest drift occurred in columns. Based on both the NBCC and ASCE definitions, the above irregularities produced soft first storeys with the first storey stiffness becoming 1/8th and half the stiffness of the storey above. The modal periods of the buildings with elevation irregularity are provided in Table 6-2.

**Table 6-2: Modal periods of regular buildings and buildings with irregularity in elevation**

No. of Storeys	Regular Buildings			Elevation Irregularity Case I			Elevation Irregularity Case II		
			Period (sec)			Period (sec)			Period (sec)
2	T1,T2	Lateral x,y	0.488	T1	Lateral x	0.520	T1	Lateral x	0.623
	T3	Torsional	0.274	T2	Lateral y	0.515	T2	Lateral y	0.497
	T4,T5	Diagonal	0.240	T3	Lateral y	0.291	T3	Torsional	0.340
				T4	Torsional	0.279	T4	Diagonal	0.253
5	T1,T2	Lateral x,y	0.824	T1	Lateral x	0.887	T1	Lateral x	1.046
	T3	Torsional	0.484	T2	Lateral y	0.825	T2	Lateral y	0.825
	T4,T5	Lateral x,y	0.424	T3	Torsional	0.512	T3	Torsional	0.544
				T4	Lateral y	0.427	T4	Lateral x	0.432
10	T1,T2	Lateral x,y	1.724	T1	Lateral x	1.862	T1	Lateral x	1.940
	T3	Torsional	1.219	T2	Lateral y	1.740	T2	Lateral y	1.740
	T4,T5	Lateral x,y	0.554	T3	Torsional	1.288	T3	Torsional	1.287
				T4	Lateral x	0.572	T4	Lateral x	0.650

**Table 6-3: Modal periods of buildings with irregularity in plan**

No. of Storeys	Torsional Irregularity Case I		Period (sec)	Torsional Irregularity Case II		Period (sec)	Torsional Irregularity Case III		Period (sec)
2	T1	Lateral x	0.690	T1	Torsional	0.876			
	T2	Lateral y	0.497	T2	Diagonal	0.628			
	T3	Torsional	0.337	T3	Torsional	0.368			
	T4	Lateral x	0.294	T4	Lateral in 45 direction	0.318			
5	T1	Lateral x	1.213	T1	Torsional	1.715	T1	Torsional	1.728
	T2	Lateral y	0.811	T2	Lateral in 45 direction	1.047	T2	Lateral in 45 direction	1.05
	T3	Torsional	0.546	T3	Torsional	0.601	T3	Torsional-Lateral	0.575
	T4	Lateral x	0.511	T4	Torsional-Lateral	0.569	T4	Diagonal	0.493
10	T1	Lateral x	2.253	T1	Torsional	2.881	T1	Torsional	2.886
	T2	Lateral y	1.719	T2	Lateral in 45 direction	2.037	T2	Lateral in 45 direction	2.049
	T3	Torsion	1.317	T3	Torsional	1.398	T3	Torsional-Lateral	0.959
	T4	Lateral x	0.769	T4	Torsional-Lateral	0.954	T4	Torsional	0.858

### 6.3. Methodology

The Computer software PERFORM 3D (CSI 2013) was used to conduct inelastic time history analysis of the selected regular and irregular shear-wall buildings. Three-dimensional numerical models were first generated using the design information. FEMA beam and FEMA column components of PERFORM 3D were employed for the frame elements with lumped plasticity in plastic hinges at member ends. Figure 6-4 illustrates the numerical model for beam and column components. Flexural capacities of members were defined using moment capacities computed by SAP2000 software (CSI 2009) which was used to conduct sectional analyses. Hinge properties were determined using ASCE 41 (ASCE 2007) recommendations for members without ductile detailing. They were assigned a bilinear relationship followed by strength decay. The initial effective stiffness was taken as  $0.35EI$  and  $0.70EI$  for beams and columns, respectively. The post yield region had zero stiffness with a negative slope defining the strength decay segment.

The shear wall elements were modelled through a fiber model for sectional flexural behaviour. The flexural behaviour was generated from integration of concrete and steel segments in the fiber model. Hognestad's un-confined concrete model (1951) was adopted for the entire wall section without confined boundary elements. Yalcin and Saatcioglu model (2000) was used to simulate longitudinal reinforcement behaviour. Nonlinear shear behaviour was modelled in the form of a stress-strain relationship to specify shear stress-shear distortion characteristics of walls. This is simulated through a trilinear shear material behaviour followed by a descending branch. The slope of the stress-strain relationship is assumed to be equal to the shear modulus "G" until the shear strength of concrete is attained. The second branch was modelled with a linear segment having a slope equal to 6% of the initial slope. This segment was then followed by a straight line as the descending branch.

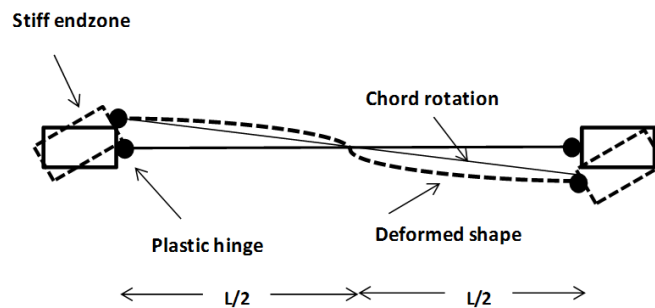


Figure 6-4: Numerical modelling of frame components

Once the analytical model was constructed, with appropriate inelastic member and material properties, then response time history analysis was conducted for a given earthquake record. Fragility analysis involves consideration of different seismic intensity levels. This was done in the current investigation through incremental dynamic analysis (Vamvatsikos and Cornell 2002a) to estimate structural performance under varying levels of seismic intensity. A set of twenty artificial earthquake ground motions,

generated by Atkinson (2009) for reference soil type C (firm soil) were selected. The records were compatible with the uniform hazard spectra (UHS) for Vancouver, specified in the 2010 National building code of Canada (2010 NBCC), and reflected western Canadian seismicity. More information on record selection is available in the Appendix A. Selected records were scaled and applied incrementally covering the entire range of structural response.

IDA involves selecting two parameters of seismic intensity measure and damage indicator. Two different seismic intensity measures were considered consisting of; i) 5% damped spectral acceleration at building period ( $S_a(T_1, 5\%)$ ), and ii) peak ground acceleration (PGA).  $S_a$  is defined in the National Building Code of Canada as a design parameter and is frequently used by designers. PGA is an intensity parameter that does not depend on building period, allowing the comparison of seismic response for regular and irregular buildings. In-plane inter-storey drift ratio of the first storey was selected in this study as the damage indicator.

IDA results provide input for developing fragility curves as a probabilistic tool representing the probability of exceeding different damage states under different levels of ground motion intensity as described in Equation 6.1.

$$P[D > Di | IM] = \phi \left[ \frac{\ln(x / Di)}{\sqrt{\beta_{d/IM}^2 + \beta_c^2 + \beta_m^2}} \right] \quad \text{Eq. 6.1}$$

In which  $\phi(\cdot)$  is the standard normal cumulative distribution function,  $Di$  is the upper bound for each damage level,  $x$  is the median value of demand as the function of IM,  $\beta_{d/IM}$  is the dispersion (logarithmic standard deviation) of the demand conditioned on the IM,  $\beta_c$  is the capacity uncertainty and  $\beta_m$  is the modeling uncertainty.

#### 6.4. Damage Quantification of Irregular Shear Wall buildings

In order to assess the seismic damage of the selected buildings, three different limit states were used; i) Immediate Occupancy, ii) Life Safety and iii) Collapse Prevention. These limit states were defined following the ASCE 41 (ASCE 2007) recommendations. Accordingly, Immediate Occupancy is the state when building remains safe to be reoccupied. Lateral-force- and gravity-load-resisting systems retain most of their design strength. The initiation of a change in the slope of an IDA curve is an indication of exceeding immediate occupancy. Life Safety is the state when the structure undergoes significant damage. The structural elements and components are severely damaged, but gravity-load-carrying elements continue fulfilling their functions. Collapse Prevention is the limit state when substantial damage is imposed on structural elements, with observation of significant strength and stiffness degradation of the lateral-load-resisting system, accompanied by large permanent lateral deformations. The structure is not repairable and not safe to reoccupy. Extensive softening in IDA curves indicates that the collapse prevention limit state has been exceeded (herein, observing 20% of the initial slope in IDA curves). The Life Safety limit state in the current study was taken as half way between Immediate Occupancy and Collapse Prevention.

The seismic assessment of buildings with plan irregularities requires quantifying damage in three dimensions. Therefore, three dimensional modelling is necessary for such assessment. Plan irregularities cause non-uniform damage among the members within a story and inter-storey drift in one direction is no longer valid to capture the localized variation in demand because of two main reasons; the drift value varies within the plan due to torsion, and high drift values are observed in two orthogonal directions even if the load is applied in one direction. In order to overcome the limitations of the conventional inter-storey damage quantification, the geometric mean of the drift values in both plan directions is considered as the damage indicator for torsionally sensitive buildings. Figure 6-5 shows an example of a building with plan irregularity case III,

where the damage index is applied to the structural member with maximum displacement. This element can be a column, and hence the damage index is not necessarily tied to the deformations experienced by the shear wall(s).

Shear wall buildings with elevation irregularities can have irregularities at different floor levels. A common case of elevation irregularities is the presence of a soft storey at the first storey level. First storeys often experience highest levels of inelasticity. Therefore, the first storey drift was adopted as a damage index.

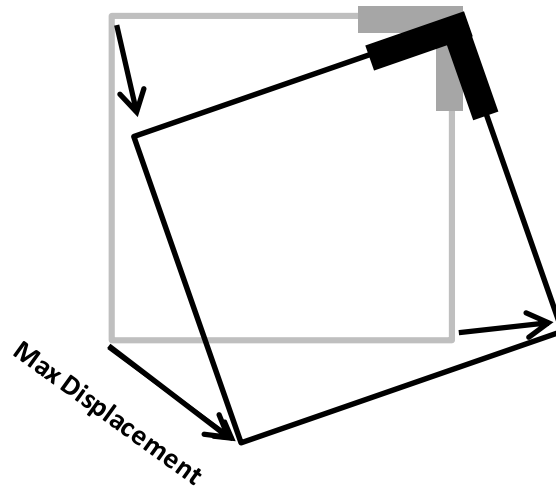


Figure 6-5: Sample definition of damage indicator for buildings with a plan irregularity

## 6.5. Fragility Curves for Irregular Shear Wall Buildings

Using the results of IDA as the input for probabilistic analysis, fragility curves were developed for shear wall buildings with plan and elevation irregularities, as shown in Figure 6-6 and Figure 6-7.

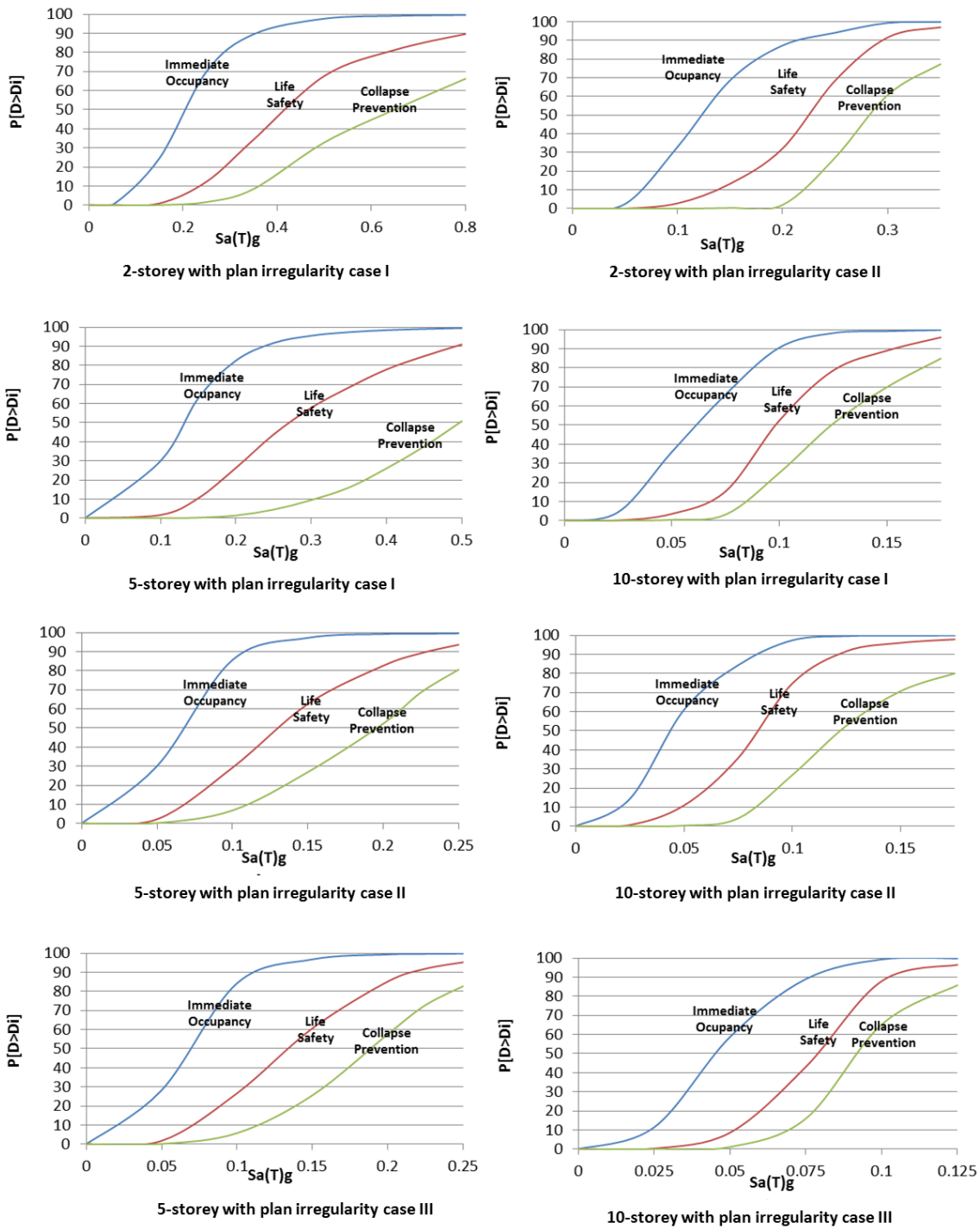


Figure 6-6: Fragility curves derived for shear wall buildings with plan irregularities

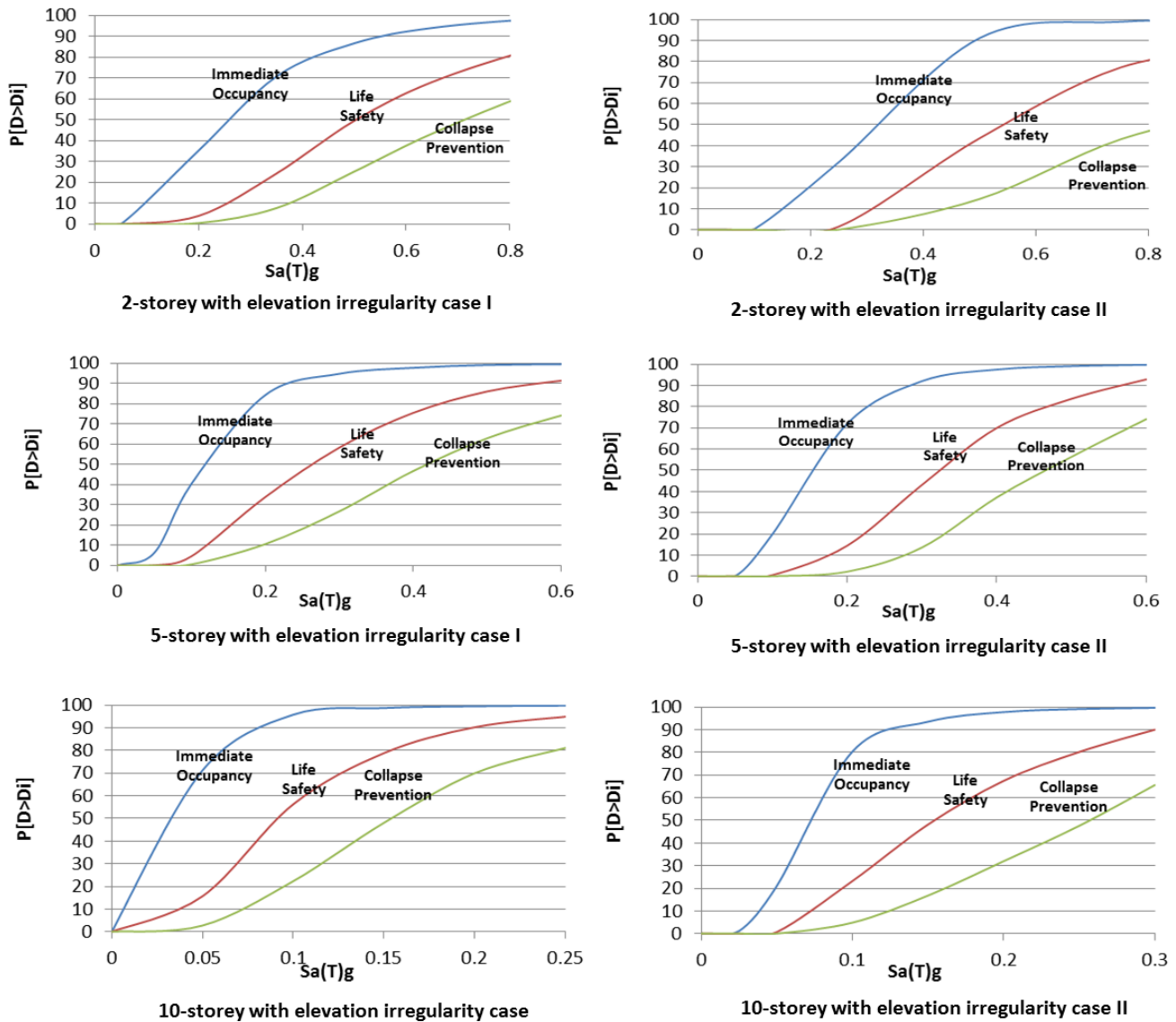


Figure 6-7: Fragility curves derived for shear wall buildings with elevation irregularities

The comparison of the effects of irregularities on fragility curves can be done more conveniently by using fragilities based on PGA, since they are not dependent on building period. Therefore, the relationships between storey drift as the damage indicator and PGA for seismic intensity are drawn for each irregular building case using the results of the IDA. This leads to a normalized platform for seismic intensity level. The results are shown in Figure 6-8 and

Figure 6-9. These graphs provide insight into the effects of building irregularities on relationships between building damage and seismic intensity. Fragility curves of buildings with different irregularities having PGA as the seismic intensity parameter are presented and compared in Figure 6-10. The results for companion regular buildings were generated in an earlier phase of the same investigation.

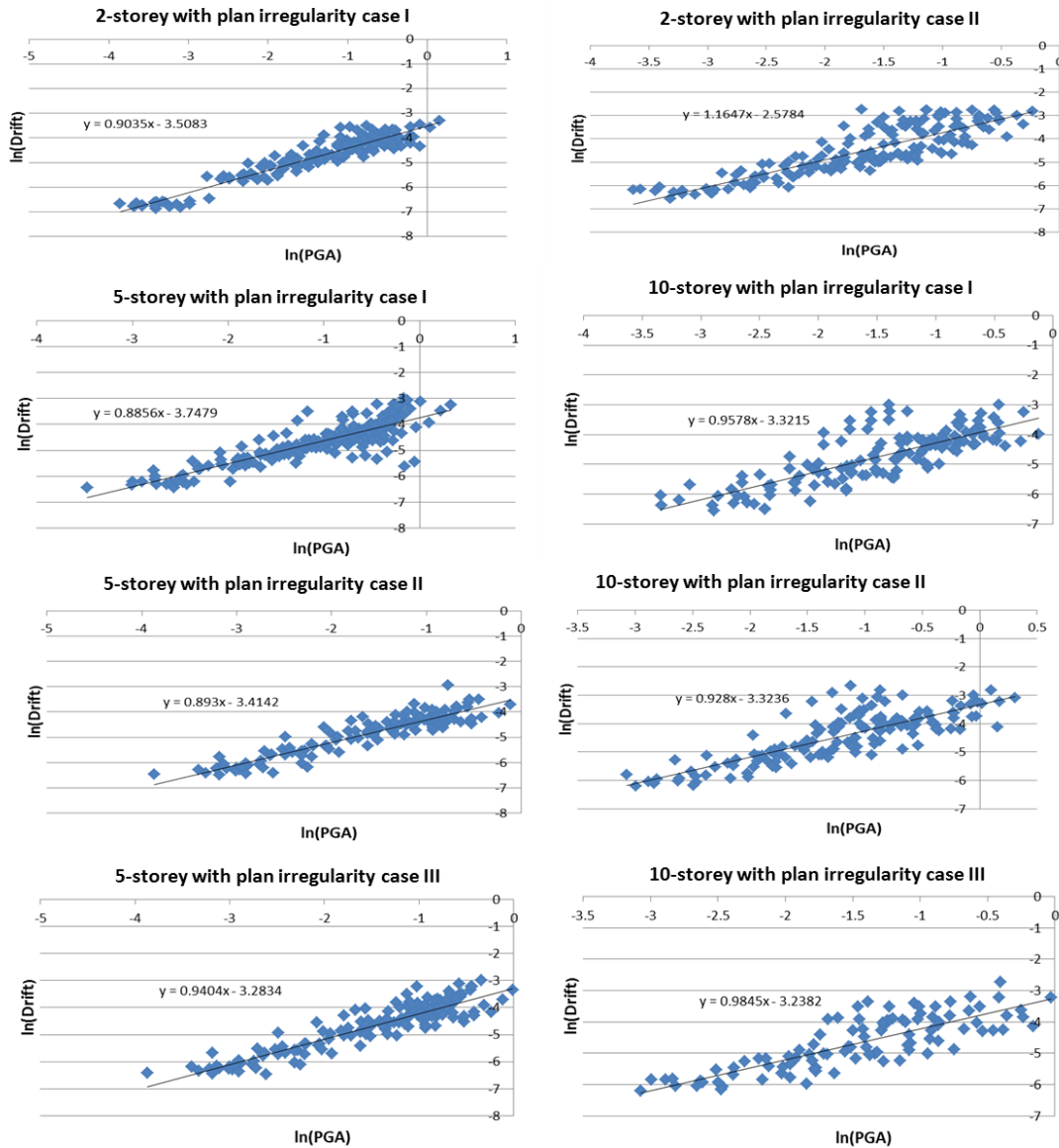


Figure 6-8: Drift-PGA relationship for shear wall buildings with plan irregularities

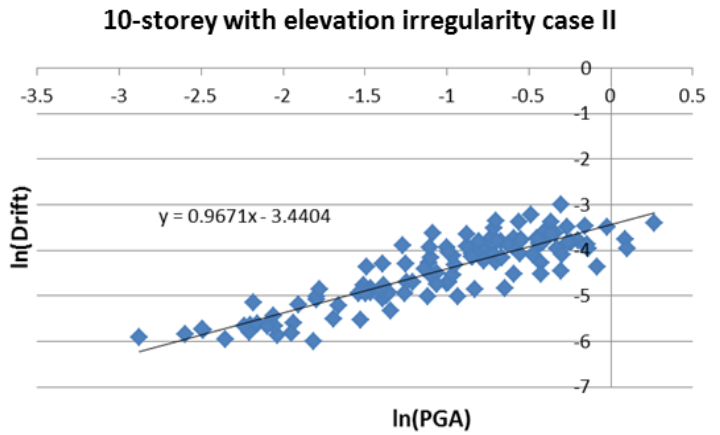
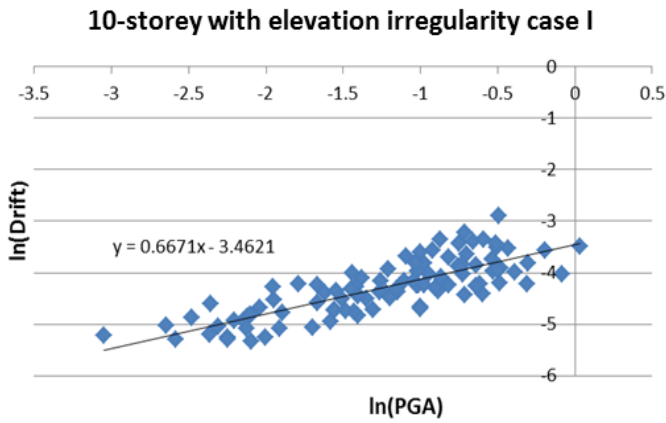
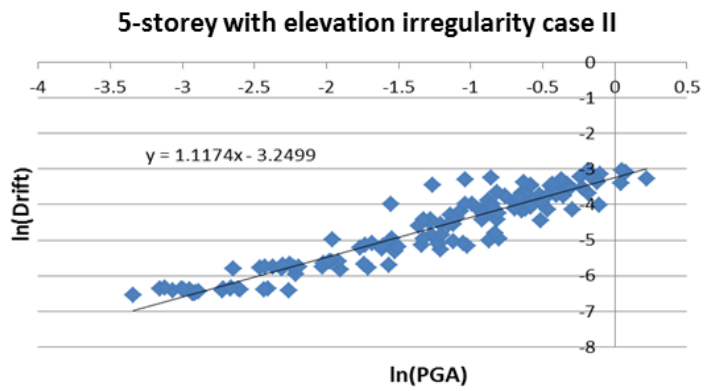
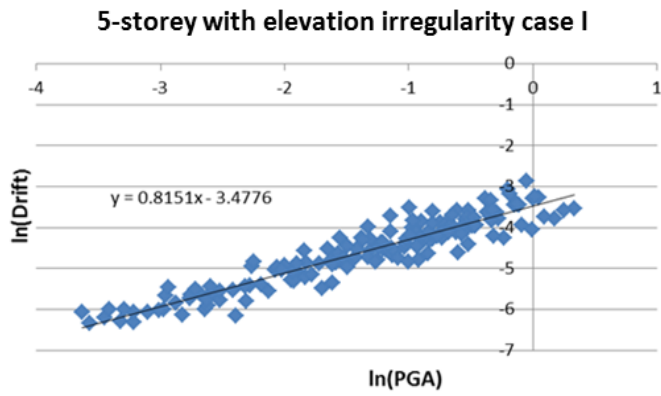
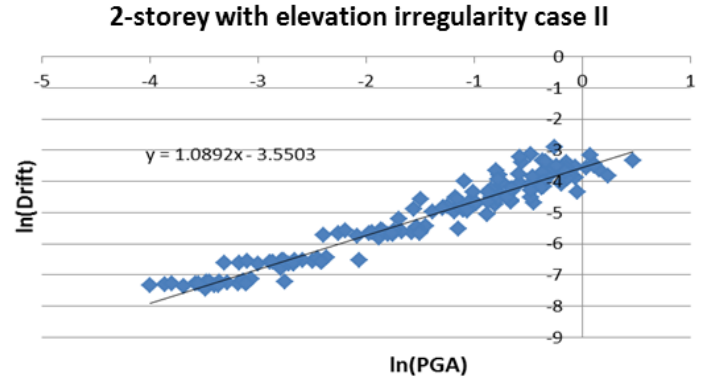
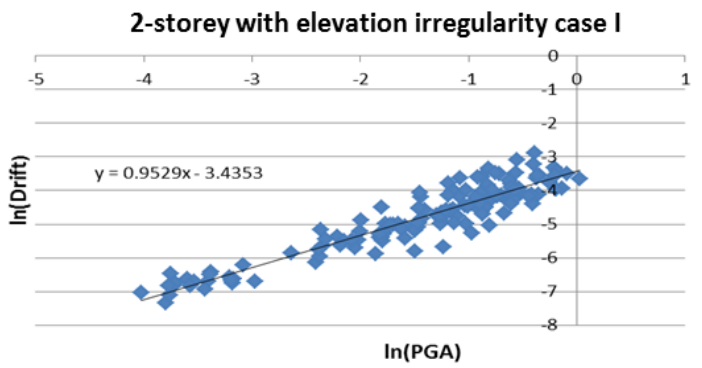


Figure 6-9: Drift-PGA relationship for shear wall buildings with elevation irregularities

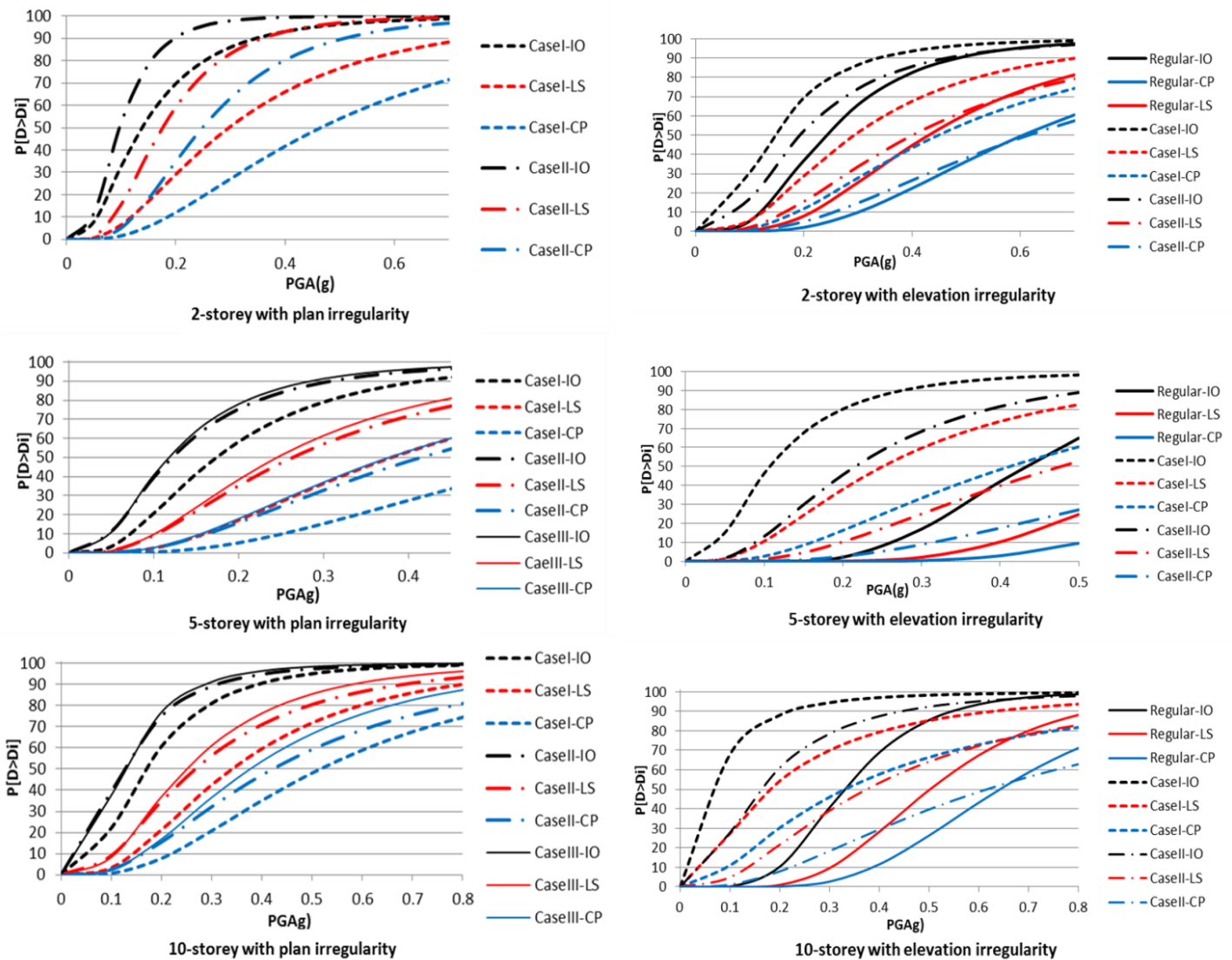


Figure 6-10: Comparison of fragility curves of buildings with different irregularities

## 6.6. Summary and Conclusions

Three dimensional building models were developed for 2-, 5-, and 10-storey reinforced concrete shear wall buildings with plan and elevation irregularities to generate seismic fragility curves. Spectral acceleration and PGA were used as two seismic intensity

measures. Storey drift was used as a damage indicator. IDA was employed to generate performance data, from which fragility curves were derived through a probabilistic approach. Three plan irregularities were considered (Case I, II and III). They correspond to buildings with  $B = 1.8$  for Case I; and  $B = 2.0$  for Cases II and III, as defined in the NBCC. In terms of the ASCE definition of torsional sensitivity, they correspond to buildings having a torsional eccentricity of 50% of the plan dimension in one direction for Case I, and having the same eccentricity in both directions for Cases II and III. The latter two cases had similar modal periods and exhibited similar behaviour. This can be attributed to the linear response of shear walls in both cases, even at the collapse inter-storey drift levels, when significant damage was incurred in corner columns. In addition, two elevation irregularity cases were considered (Case I and II), representing soft storeys at the first-storey level with  $1/8$  and  $1/2$  of the stiffness of the storey above.

Figure 6-10 gives a summary of seismic fragility analysis of buildings with different heights and irregularities. It indicates that the probability of exceeding different damage states increases with irregularities. At  $PGA = 0.2$  g, regular buildings of all heights meet the life safety criterion with only the 2-storey building exceeding it by a probability of 7%. In contrast, buildings with plan irregularities showed 20% to 60% probabilities of exceedance with the 2-storey building being more vulnerable than the taller buildings. Similarly, the elevation irregularities indicate probability of exceedance of the life safety limit by 10% to 50%, but this time the 10-storey building shows higher vulnerability than the buildings with lower heights. As the intensity increases to  $PGA = 0.4$ g, the regular buildings show 10% to 40% probabilities of exceedance of life safety, whereas buildings with irregularities show approximately 40% to 80% probabilities of exceedance, except for plan irregularities in the 2-storey building, which show 70% to 90% probabilities of exceeding the life safety criterion.

Table 6-4 summarizes the probabilities of exceeding different damage states at 2010 NBCC level of PGA for Vancouver. Figure 6-11 shows the increase in probability

of exceeding each damage state for irregular buildings compared to their regular counterparts for PGA values of 0.1 to 0.4 with 0.1 increments. The figure indicates that the Plan Irregularity Case II shows more building vulnerability relative to Case I and Elevation Irregularity Case I shows more vulnerability relative to Case II.

It may be concluded that the fragility curves derived in the current research project provide a convenient resource for engineers to assess the effects of building irregularities on seismic damage. While the existing older regular shear wall buildings may pose threat to public safety, irregularities in these buildings further increase their seismic vulnerabilities. The fragility curves developed may be used for seismic risk mitigation strategies with due considerations given to hazard levels and performance objectives.

**Table 6-4: Probability of exceedance for each damage state at 2010NBCC load level**

Building	Limit State	Plan Irregularity		Elevation Irregularity	
		Case I	Case II	Case I	Case II
2-Storey	IO	95%	100%	95%	90%
	LS	75%	95%	75%	55%
	CP	50%	85%	50%	35%
5-Storey	IO	90%	100%	98%	90%
	LS	60%	80%	80%	50%
	CP	35%	60%	60%	25%
10-Storey	IO	95%	98%	98%	95%
	LS	65%	85%	85%	65%
	CP	45%	65%	65%	40%

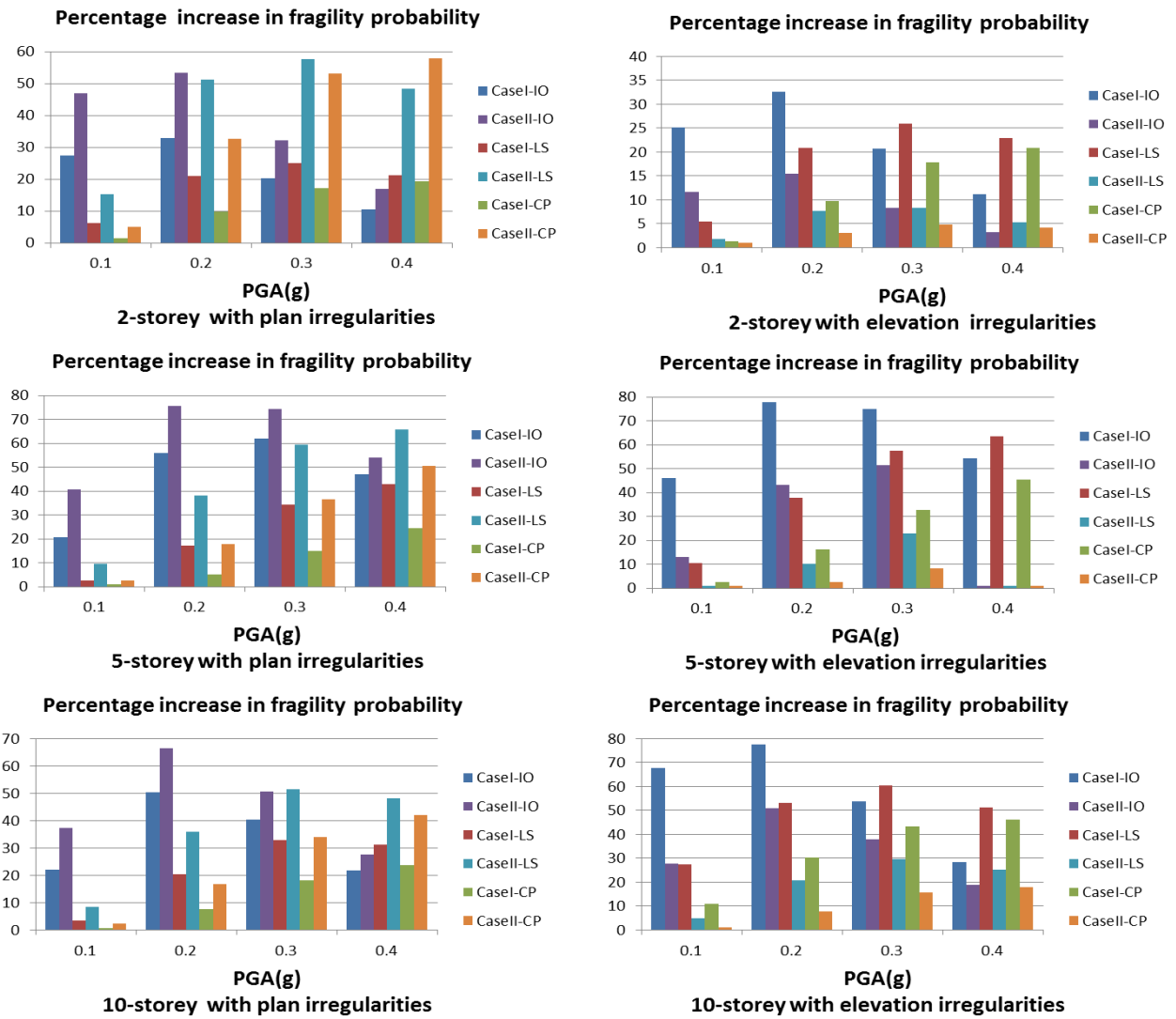


Figure 6-11: Increase in seismic fragility probabilities due to building irregularities relative to companion regular buildings

## Chapter 7. Summary and Conclusion

Seismic fragility analysis was conducted for reinforced concrete shear-wall buildings in Canada. Three building heights (2; 5; and 10 stories) and two geographic locations (Vancouver in a region of high seismicity and Ottawa in a region of medium seismicity) were considered. Buildings with a regular structural layout as well as plan and elevation irregularities were designed using the seismic requirements of the National Building Code of Canada and analyzed using code-compatible synthetic earthquake records. The buildings were designed either as representatives of pre-1975 era, as conventional non-ductile buildings, or post-1975 era with ductile design and detailing principles. Computer software PERFORM-3D was selected for dynamic inelastic response time history analysis. The buildings were modelled with appropriate hysteretic relationships incorporating strength and stiffness degradations. The analytical model for hysteretic relationship of walls was verified extensively against test data. Incremental Dynamic Analysis was employed for structural evaluation under incrementally increasing earthquake intensity. Both spectral acceleration and peak ground acceleration were considered as intensity measures. First storey drift was selected as a damage parameter, and three performance levels were adopted; immediate occupancy, life safety and collapse prevention. The results of incremental dynamic analysis were used to develop probabilistic seismic assessment tools in the form of fragility curves. These curves were developed for each building, in each location and for each design (with regular and

irregular structural layouts), covering different years of construction and wide spectrum of buildings in practice. The fragility curves, providing the probability of exceeding different damage states selected as target performance levels, were then used to assess seismic vulnerability of buildings.

The following conclusions can be drawn from the analytical investigation reported in this thesis, based on the seismic hazard values specified in the 2010 NBCC (2475 years seismic events which indicate 2% in 50 years probability):

- Fragility curves generated for representative shear wall buildings selected from different categories of buildings with different structural layouts, located in different geographic locations with different seismic hazards and years of construction can be used for probabilistic assessment of seismic damage. The suite of fragility curves presented in this thesis provides a powerful set of analytical tools for seismic vulnerability assessment of shear wall buildings in Canada.
- The 2-storey and 5-storey buildings designed for Vancouver following the requirements of the 2010 NBCC, when assessed based on spectral accelerations at empirical code periods meet the life safety performance limit, exhibiting 5% and 10% probability of exceedance. The 10-storey building show higher probability of exceedance of 50%. For the same intensity level, the 2, 5 and 10-storey buildings show 25%, 50% and 85% probability of exceeding the immediate occupancy performance limit, respectively.
- The 2-storey and 5-storey buildings in Ottawa, designed based the 2010 NBCC, indicate 0% probability of exceeding life safety, while the 10-storey building show 3% of probability at code spectral accelerations corresponding to empirical

periods. The probabilities of exceeding immediate occupancy limit state for the same three buildings in Ottawa are 10%, 5% and 25%, respectively.

- Buildings designed based on post-1975, but pre-2005 editions of NBCC for Vancouver and Ottawa indicate higher probabilities of exceedance at each damage stage when compared with the 2010 NBCC designs. 2, 5, and 10-storey buildings designed based on the earlier editions of the code in Ottawa indicate 2%, 3%, and 10% probabilities of exceeding life safety at code spectral accelerations corresponding to empirical periods, while the 2-storey building designed for Vancouver exhibited 20% probability of exceeding the life safety limit state.
- The buildings designed on the basis of the 2010 NBCC for Vancouver, when assessed under increased intensity level of 1% in 50 year earthquakes indicate 30%, 35% and 55% probability of exceeding life safety for 2, 5, and 10-storey buildings, respectively. The same buildings indicate 70%, 80%, and 90% probabilities of exceeding immediate occupancy and 10%, 10%, and 30% probabilities of exceeding collapse prevention limit states, respectively.
- The buildings designed based on 2010 NBCC for Ottawa with the same storey heights indicate 30%, 27%, and 18% probabilities of exceeding immediate occupancy and 7%, 2%, and 2% probabilities of exceeding life safety limit states under seismic events with 1% in 50 years probabilities. These buildings show a safe margin for collapse prevention limit states.
- The buildings designed in pre-seismic code era, that is based on the 1965 NBCC for Vancouver and the 1970 NBCC for Ottawa, show higher vulnerabilities compared with those designed based on the 2010 NBCC. Under spectral accelerations at empirical periods corresponding to 2% in 50 years hazard level, the buildings designed prior to 1975 in Vancouver indicate 15%, 45%, 100%

probabilities of exceeding the life safety limit states. The same buildings show 60%, 75% and 100% probabilities of exceeding immediate occupancy. The 5-storey and 10-storey buildings demonstrate 30%, and 90% probabilities of exceeding the collapse prevention limit state, while the 2-storey building indicates 2% probability of exceedance for the same limit state. The buildings designed in Ottawa prior to 1975 indicate 10%, 30%, and 65% probabilities of exceeding immediate occupancy; and 0%, 10%, and 35% probabilities of exceeding life safety limit state.

- The probability of exceeding different damage states increases with increase in the degree of irregularity. Plan Irregularity Case II (B=2) buildings demonstrate higher seismic vulnerability relative to Case I (B=1.8). Elevation Irregularity Case I buildings (with soft storey stiffness of 1/8 of storey above) indicate higher seismic vulnerability relative to Case II (with soft storey stiffness of 1/2 of storey above).
- When PGA is used as seismic intensity measure with 2% in 50 years probability, buildings with Case I plan irregularity in Vancouver indicate 75%, 60% and 65% probabilities of exceeding life safety limit state for 2, 5, and 10-storey heights, respectively. The same buildings show 50%, 35%, and 45% probabilities of exceeding collapse prevention. Case II plan irregularity leads to 95%, 80% and 85% probabilities of exceeding life safety for the same building heights. These buildings show 85%, 60%, and 65% probabilities of exceeding collapse prevention. None of the buildings with plan irregularities fall in the immediate occupancy state of performance.
- When PGA is used as seismic intensity measure with 2% in 50 years probability, buildings with Case I elevation irregularity in Vancouver demonstrate 75%, 80%, and 85% probabilities of exceeding the life safety limit state for 2, 5 and 10,

storey heights. The same buildings show 75%, 80%, and 85% probabilities of exceeding collapse prevention. When Case II elevation irregularity is considered, the buildings indicate 55%, 50%, and 65% probabilities of exceeding life safety for 2, 5 and 10, storey heights, respectively. The same buildings develop 85%, 60%, and 65% probabilities of exceeding the collapse prevention limit state. None of the buildings with elevation irregularities fall in the immediate occupancy state of performance.

- Building vulnerability is affected significantly by the level of seismic intensity. The examination of results for pre-1975 buildings in Vancouver indicates that, at  $PGA = 0.2$  g, the regular buildings of all heights meet the life safety criterion with only 2-storey building exceeding it by a probability of 7%. In contrast, buildings with plan irregularities showed 20% to 60% probabilities of exceedance with 2-storey building being more vulnerable than the taller buildings. At  $PGA = 0.4g$ , the regular buildings show 10% to 40% probabilities of exceeding life safety, whereas buildings with irregularities show approximately 40% to 80% probabilities of exceedance, except for plan irregularities in the 2-storey building, which show 70% to 90% probabilities of exceeding the life safety criterion.

Following recommendations are made for future research:

- High-rise shear wall buildings having more than 10-storey height were not considered in the study. Further development can be undertaken for high-rise shear wall buildings with more than 10-stories.
- The shear wall buildings designed prior to 1953 were not considered in the current investigation. New fragility curves can be developed for buildings designed prior to 1953 with due considerations given to the design load levels and design practices of the era.

- Other types of irregularities defined in NBCC can be considered as part of future research work, and additional seismic fragility curves can be developed for these irregular buildings.
- Buildings with seismic vulnerabilities may need to be retrofitted to reduce the risk level based on intended performance objectives. Additional fragility analysis can be conducted using retrofitted buildings as part of future research.

## References

- ACI. 1956. *Building code requirements for reinforced concrete (ACI 318-56)*: American Concrete Institute.
- ACI. 1963. *Building code requirements for reinforced concrete (ACI 318-63)*: American Concrete Institute.
- ACI. 1971. *Building code requirements for reinforced concrete (ACI318-71)*: American Concrete Institute.
- ACI. 2005. *Building code requirements for structural concrete (ACI 318-05) and commentary (ACI 318R-05)*: American Concrete Institute.
- Adams, John, and Gail Atkinson. 2003. "Development of seismic hazard maps for the proposed 2005 edition of the National Building Code of Canada." *Canadian Journal of Civil Engineering* no. 30 (2):255-271.
- Adebar, Perry, James Mutrie, and Ronald DeVall. 2005. "Ductility of concrete walls: the Canadian seismic design provisions 1984 to 2004." *Canadian Journal of Civil Engineering* no. 32 (6):1124-1137.
- Al-Sadoon, Zaid. 2016. *Seismic Retrofitting of Conventional Reinforced Concrete Moment-Resisting Frames Using Buckling Restrained Braces*, Université d'Ottawa/University of Ottawa.
- ASCE. 2007. *Seismic rehabilitation of existing buildings*. ASCE 41 Reston, VA American Society of Civil Engineers.
- ASCE. 2013. *Seismic rehabilitation of existing buildings*. ASCE 41 Reston, VA American Society of Civil Engineers.
- ATC. 1997. *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings*, prepared for the Building Seismic Safety Council/Federal Emergency Management Agency, FEMA 274. Washington, D.C Applied Technology Council
- Athanassiadou, CJ. 2008. "Seismic performance of R/C plane frames irregular in elevation." *Engineering structures* no. 30 (5):1250-1261.
- Atkinson, Gail M. 2004. An overview of developments in seismic hazard analysis. Paper read at 13th World conference on earthquake engineering. Vancouver, BC, Canada, August.
- Atkinson, Gail M. 2009. "Earthquake time histories compatible with the 2005 National building code of Canada uniform hazard spectrum." *Canadian Journal of Civil Engineering* no. 36 (6):991-1000.

- Beigi, Hossein Agha, Timothy John Sullivan, Constantin Christopoulos, and Gian Michele Calvi. 2015. "Factors influencing the repair costs of soft-story RC frame buildings and implications for their seismic retrofit." *Engineering Structures* no. 101:233-245.
- Belletti, Beatrice, Cecilia Damoni, and Antonello Gasperi. 2013. "Modeling approaches suitable for pushover analyses of RC structural wall buildings." *Engineering Structures* no. 57:327-338.
- Bilgin, Huseyin. 2013. "Fragility-based assessment of public buildings in Turkey." *Engineering Structures* no. 56:1283-1294.
- Buratti, N, B Ferracuti, and M Savoia. 2010. "Response surface with random factors for seismic fragility of reinforced concrete frames." *Structural Safety* no. 32 (1):42-51.
- Celik, Ozan Cem, and Bruce R Ellingwood. 2009. "Seismic risk assessment of gravity load designed reinforced concrete frames subjected to Mid-America ground motions." *Journal of structural engineering* no. 135 (4):414-424.
- Celik, Ozan Cem, and Bruce R. Ellingwood. 2010. "Seismic fragilities for non-ductile reinforced concrete frames—Role of aleatoric and epistemic uncertainties." *Structural Safety* 32, no. 1: 1-12.
- Cornell, C Allin. 1968. "Engineering seismic risk analysis." *Bulletin of the Seismological Society of America* no. 58 (5):1583-1606.
- Cornell, C Allin, and Helmut Krawinkler. 2000. "Progress and challenges in seismic performance assessment." *PEER Center News* no. 3 (2):1-3.
- CSA. 1959. Design of concrete structures, CSA A23. 3–59. In *CSA, Rexdale, Ontario*: Canadian Standards Association.
- CSA. 1973. Design of concrete structures, CSA A23. 3–73. In *CSA, Rexdale, Ontario*: Canadian Standards Association.
- CSA. 1977. Design of concrete structures, CSA A23. 3–77. In *CSA, Rexdale, Ontario*: Canadian Standards Association.
- CSA. 1984. Design of concrete structures, CSA A23. 3–84. In *CSA, Rexdale, Ontario*: Canadian Standards Association.
- CSA. 1994. Design of concrete structures, CSA A23. 3–94. In *CSA, Rexdale, Ontario*: Canadian Standards Association.
- CSA. 2004. Design of concrete structures, CSA A23. 3–04. In *CSA, Rexdale, Ontario*: Canadian Standards Association.
- CSI. 2009. SAP. Integrated Software for Structural Analysis and Design. Berkeley, CA: Computers & Structures, Inc.

- CSI. 2013. Perform 3D Version 5.0.1. Nonlinear Analysis and Performance Assessment of 3D Structures. Berkeley, CA: Computers & Structures, Inc.
- EarthquakesCanada. 2016. <http://www.earthquakescanada.nrcan.gc.ca/>. 22/07/2016 ed: National Research Council of Canada.
- El Sabbagh, Amid. 2014. *Seismic Risk Assessment of Unreinforced Masonry Buildings Using Fuzzy Based Techniques for the Regional Seismic Risk Assessment of Ottawa, Ontario*, Université d'Ottawa/University of Ottawa.
- Ellingwood, Bruce R, Ozan Cem Celik, and Kursat Kinali. 2007. "Fragility assessment of building structural systems in Mid-America." *Earthquake Engineering & Structural Dynamics* no. 36 (13):1935-1952.
- Eurocode. 2005. British Standard, Eurocode 8: Design of structures for earthquake resistance.
- FEMA-273. 1997. NEHRP guidelines for the seismic rehabilitation of buildings. In *FEMA-273, Federal Emergency Management Agency, Washington, DC*.
- FEMA. 2003. HAZUS MR4 Technical Manual. Whashington (DC): Department of Homeland Security, Federal Emergency Management Agency, Mitigation Division.
- Ghodsi, Tony, and Jose A Flores Ruiz. 2010. "Pacific earthquake engineering research/seismic safety commission tall building design case study 2." *The Structural Design of Tall and Special Buildings* no. 19 (1-2):197-256.
- Gokdemir, H, H Ozbasaran, M Dogan, E Unluoglu, and U Albayrak. 2013. "Effects of torsional irregularity to structures during earthquakes." *Engineering Failure Analysis* no. 35:713-717.
- Haselton, Curt B, Abbie B Liel, Gregory G Deierlein, Brian S Dean, and Jason H Chou. 2010. "Seismic collapse safety of reinforced concrete buildings. I: Assessment of ductile moment frames." *Journal of Structural Engineering* no. 137 (4):481-491.
- Hognestad, Eivind. 1951. "Study of combined bending and axial load in reinforced concrete members." *University of Illinois. Engineering Experiment Station. Bulletin; no. 399*.
- IBC, ICC. 2012. "International building code." *International Code Council, Inc*.
- Jeon, Jong-Su, Laura N Lowes, Reginald DesRoches, and Ioannis Brilakis. 2015. "Fragility curves for non-ductile reinforced concrete frames that exhibit different component response mechanisms." *Engineering Structures* no. 85:127-143.
- Jeong, Seong-Hoon, and Amr S Elnashai. 2007. "Fragility relationships for torsionally-imbalanced buildings using three-dimensional damage characterization." *Engineering structures* no. 29 (9):2172-2182.

- Jeong, Seong-Hoon, Aman M Mwafy, and Amr S Elnashai. 2012. "Probabilistic seismic performance assessment of code-compliant multi-story RC buildings." *Engineering Structures* no. 34:527-537.
- Ji, Jun, Amr S Elnashai, and Daniel A Kuchma. 2007. "An analytical framework for seismic fragility analysis of RC high-rise buildings." *Engineering Structures* no. 29 (12):3197-3209.
- Ji, Jun, Amr S Elnashai, and Daniel A Kuchma. 2009. "Seismic fragility relationships of reinforced concrete high-rise buildings." *The Structural Design of Tall and Special Buildings* no. 18 (3):259-277.
- Karapetrou, ST, SD Fotopoulou, and KD Pitilakis. 2015. "Seismic vulnerability assessment of high-rise non-ductile RC buildings considering soil-structure interaction effects." *Soil Dynamics and Earthquake Engineering* no. 73:42-57.
- Kent, Dudley Charles, and Robert Park. 1971. "Flexural members with confined concrete." *Journal of the Structural Division* no. 97 (7):1969-1990.
- Koduru, SD, and T Haukaas. 2009. "Probabilistic seismic loss assessment of a Vancouver high-rise building." *Journal of structural engineering* no. 136 (3):235-245.
- Lupoi, Giorgio, Paolo Franchin, Alessio Lupoi, and Paolo E Pinto. 2006. "Seismic fragility analysis of structural systems." *Journal of Engineering Mechanics* no. 132 (4):385-395.
- Masi, Angelo, and Marco Vona. 2012. "Vulnerability assessment of gravity-load designed RC buildings: Evaluation of seismic capacity through non-linear dynamic analyses." *Engineering Structures* no. 45:257-269.
- Mitchell, Denis, Patrick Paultre, Rene Tinawi, Murat Saatcioglu, Robert Tremblay, Kenneth Elwood, John Adams, and Ronald DeVall. 2010. "Evolution of seismic design provisions in the National building code of Canada." *Canadian Journal of Civil Engineering* no. 37 (9):1157-1170.
- Mitropoulou, Chara Ch, and Manolis Papadrakakis. 2011. "Developing fragility curves based on neural network IDA predictions." *Engineering Structures* no. 33 (12):3409-3421.
- Mohammadi-Doostdar, Hossein. 1994. *Behaviour and design of earthquake resistant low-rise shear walls*: University of Ottawa (Canada).
- NRCC. 1941. National Building Code of Canada. Ottawa, Canada: National Research Council of Canada.
- NRCC. 1960. National Building Code of Canada. Ottawa, Canada: National Research Council of Canada.
- NRCC. 1965. National Building Code of Canada. Ottawa, Canada: National Research Council of Canada.
- NRCC. 1970. National Building Code of Canada. Ottawa, Canada: National Research Council of Canada.
- NRCC. 1975. National Building Code of Canada. Ottawa, Canada: National Research Council of Canada.

- NRCC. 1980. National Building Code of Canada. Ottawa, Canada: National Research Council of Canada.
- NRCC. 1985. National Building Code of Canada. Ottawa, Canada: National Research Council of Canada.
- NRCC. 1990. National Building Code of Canada. Ottawa, Canada: National Research Council of Canada.
- NRCC. 1995. National Building Code of Canada. Ottawa, Canada: National Research Council of Canada.
- NRCC. 2005. National Building Code of Canada. Ottawa, Canada: National Research Council of Canada.
- NRCC. 2010. National Building Code of Canada. Ottawa, Canada: National Research Council of Canada.
- NZS3101. 1982. Code of Practice for the Design of Concrete Structures.
- Oesterle, RG. 1976. Earthquake resistant structural walls: Tests of isolated walls. Research and Development Construction Technology Laboratories, Portland Cement Association.
- Pejovic, Jelena, and Srdjan Jankovic. 2015. "Seismic fragility assessment for reinforced concrete high-rise buildings in Southern Euro-Mediterranean zone." *Bulletin of Earthquake Engineering*:1-28.
- Pitilakis, KD, ST Karapetrou, and SD Fotopoulou. 2014. "Consideration of aging and SSI effects on seismic vulnerability assessment of RC buildings." *Bulletin of earthquake engineering* no. 12 (4):1755-1776.
- Rajeev, Pathmanathan, and S Tesfamariam. 2012a. "Seismic fragilities for reinforced concrete buildings with consideration of irregularities." *Structural Safety* no. 39:1-13.
- Rajeev, Pathmanathan, and S Tesfamariam. 2012b. "Seismic fragilities of non-ductile reinforced concrete frames with consideration of soil structure interaction." *Soil Dynamics and Earthquake Engineering* no. 40:78-86.
- Ramamoorthy, Sathish K, Paolo Gardoni, and Joseph M Bracci. 2006. "Probabilistic demand models and fragility curves for reinforced concrete frames." *Journal of structural Engineering* no. 132 (10):1563-1572.
- Ramamoorthy, Sathish K, Paolo Gardoni, and Joseph M Bracci. 2008. "Seismic fragility and confidence bounds for gravity load designed reinforced concrete frames of varying height." *Journal of structural engineering* no. 134 (4):639-650.
- Saatcioglu, Murat, and Salim R Razvi. 1992. "Strength and ductility of confined concrete." *Journal of Structural Engineering*.
- Sarkar, Pradip, A Meher Prasad, and Devdas Menon. 2010. "Vertical geometric irregularity in stepped building frames." *Engineering Structures* no. 32 (8):2175-2182.
- Scott, Michael H, and Gregory L Fenves. 2006. "Plastic hinge integration methods for force-based beam-column elements." *Journal of Structural Engineering*.

- SEAOC. 1995. "Performance based seismic engineering of buildings." *Structural Engineers Association of California*.
- UBC. 1935. Uniform Building Code 1935. Long Beach, California: International Conference of Building Officials (ICBO).
- Vamvatsikos, Dimitrios, and C Allin Cornell. 2002a. "Incremental dynamic analysis." *Earthquake Engineering & Structural Dynamics* no. 31 (3):491-514.
- Vamvatsikos, Dimitrios, and C Allin Cornell. 2002b. The incremental dynamic analysis and its application to performance-based earthquake engineering. Paper read at Proceedings of the 12th European Conference on Earthquake Engineering.
- Vamvatsikos, Dimitrios, and C Allin Cornell. 2004. "Applied incremental dynamic analysis." *Earthquake Spectra* no. 20 (2):523-553.
- Varadharajan, S, VK Sehgal, and Babita Saini. 2013. "Determination of inelastic seismic demands of RC moment resisting setback frames." *Archives of Civil and Mechanical Engineering* no. 13 (3):370-393.
- Ventura, Carlos E, WD Liam Finn, Tuna Onur, Ardel Blanquera, and Mahmoud Rezai. 2005. "Regional seismic risk in British Columbia-classification of buildings and development of damage probability functions." *Canadian Journal of Civil Engineering* no. 32 (2):372-387.
- Wallace, John W. 2012. "Behavior, design, and modeling of structural walls and coupling beams—Lessons from recent laboratory tests and earthquakes." *International Journal of Concrete Structures and Materials* no. 6 (1):3-18.
- Wen, Y. K., Bruce Russell Ellingwood, and Joseph M. Bracci. 2004. "Vulnerability function framework for consequence-based engineering." *MAE Center Report 04-04*.
- Yalcin, C, and M Saatcioglu. 2000. "Inelastic analysis of reinforced concrete columns." *Computers & Structures* no. 77 (5):539-555.
- Zareian, Farzin, and Helmut Krawinkler. 2010. "Structural system parameter selection based on collapse potential of buildings in earthquakes." *Journal of structural engineering* no. 136 (8):933-943.

# Appendix A: Site Specific Record Selection and Seismic Fragility Analysis

**Abstract:** Seismic fragility of a 5-storey regular shear wall building was derived through nonlinear dynamic analysis and the effect of site specific record selection and direction of applied load were investigated. Two sets of twenty synthetic earthquake records compatible with western and eastern Canadian seismicity are applied to the structural model. Fragility curves of the building were developed having spectral acceleration at the fundamental period of the structure and peak ground acceleration as seismic intensity measures. The results indicated that fragility curves are less sensitive to record selection when spectral acceleration is chosen as the seismic intensity measure, as this parameter provides a normalized format of hazard intensity. On the other hand, when peak ground acceleration is chosen as seismic intensity indicator, significant change is observed in fragility curves derived for sites with different seismicity characteristics. Numerical simulation of the building was further analyzed under western seismic records applied at an angle of 45 degree and the results of dynamic analysis and fragility curves compared to those obtained when the records were applied parallel to one of the principle directions. Based on the results, application of bidirectional load would cause a delay in entering nonlinear range of behaviour for the structure and therefore is not a conservative approach at lower levels of seismic intensity.

## Introduction

Damage observed after past earthquakes prove that many existing buildings are vulnerable when subjected to strong earthquakes. Therefore seismic performance of existing buildings needs to be assessed to decide on the necessity of retrofitting. Seismic vulnerability of buildings can be assessed through detailed nonlinear modelling and dynamic analysis. Before performing numerous time consuming analyses for several building cases, potential parameters affecting the results must be investigated to obtain results efficiently. Herein, two aspects of dynamic analysis that potentially contribute to the results of vulnerability analysis are studied; site specific record selection and direction of applied load.

Seismic assessment of buildings requires choosing earthquake records representative of seismicity of the region. If sufficient number of past earthquake records are not available, synthetic records compatible with site seismic characteristic are to be selected. Among previous research studies on seismic assessment of concrete buildings are those by Ramamoorthy et al. (2006) and Celik and Ellingwood (2009) who derived fragility curves of two-dimensional frames using synthetic ground motions developed for the Memphis region (mid-America) with spectral acceleration at the fundamental period of the structure as hazard intensity parameter. Koduru and Haukaas (2009) modeled a 15 storey shear wall building using OpenSees in 3D and assessed the behaviour of the building under three different types of ground motions (crustal, sub-crustal and subduction zone). Zareian, and Krawinkler (2010) studied the collapse capacity of moment-resisting frame and shear wall structures applying Incremental Dynamic Analysis in Drain 2D. Instead of using scalar value of intensity measure (such as  $S_a$ ), vectored-value of  $(S_a, \epsilon)$  was used, where  $\epsilon$  stands for the difference between  $S_a$  of a specific ground motion and the median of the spectral acceleration predicted by an attenuation relationship at the fundamental period of the structure. Another study using Perform 3D software was conducted by Pejovic and Jankovic(2015) to derive seismic

fragility of reinforced concrete high rise-buildings in Southern euro-Mediterranean zone with core wall structural system choosing 60 ground motions with a wide range of magnitudes, distances to source and different site conditions.

Regarding direction of applied load, ASCE 7-10 (ASCE 2010) standard on minimum design loads requires applying time history records in both horizontal directions in 3D analysis, while there is no such requirement in Canadian Codes (NRCC 2010) . In current investigation, the purpose is seismic assessment of building rather than design. Accordingly, a 5-storey shear wall building having 5 bays of 7.0 m length and 4.0 m storey height was selected as a reference building to analyze the effect of parameters mentioned. The building was designed based on 1990NBCC (NRCC 1990) load level for Ottawa. The building was analyzed under synthetic seismic records compatible with western and eastern Canadian seismicity to investigate the effect of site specific record selection. The effect of direction of the applied seismic load was also investigated by considering two cases; applying time history records in one of principle directions and applying them at 45 degree.

### **Site Specific Time History Records**

In order to perform incremental dynamic analysis, a set of ground motion records representative of the building site are needed. Herein, artificial earthquake ground motions, generated by Atkinson (2009), were used. These records are compatible with the uniform hazard spectra (UHS) specified for seismic design in the 2005 and 2010 National building code of Canada. The records were generated for earthquakes having 2% probability of exceedance in 50 years. The target UHS depends on the location and the site condition, where the site condition is classified based on the time-averaged shear-wave velocities in the top 30 m of soil deposit (soil types A, C, D, and E specified in the building code NRCC 2010). Atkinson applied the stochastic finite-fault method to generate earthquake time histories matching the 2005 NBCC UHS for a range of

Canadian sites and different soil types. In this study, the records generated for the reference soil type C were used.

The records are provided in four sets of 45 time histories: M6.5 at 10 to 15 km, M6.5 at 20 to 30 km, M7.5 at 15 to 25 km, and M7.5 at 50 to 100 km for Western Canada and four sets of 45 time histories: M6.5 at 10 to 15 km, M6.5 at 20 to 30 km, M7.5 at 15 to 25 km, and M7.5 at 50 to 100 km for Eastern Canada. 5 records were selected from each of these eight groups (twenty records for each site), which matched the target spectrum in the period range of 0.5 to 2.5 for east and west. These records with lowest standard deviation with respect to target spectrum in the range of periods of interests were selected (minimum standard deviation for  $(S_a)_{\text{target}}/(S_a)_{\text{simulated}}$ ). Figure A1 shows the comparison between the uniform hazard spectrum (UHS) of Vancouver (western Canada) and Ottawa (eastern Canada) and spectral accelerations of selected records for each city while the records are scaled to the level of UHS.

## **Modelling and Analysis**

A five storey shear wall building was selected as the representative building. Three dimensional nonlinear model of the shear wall building was simulated in PRFORM 3D (CSI 2013). Beam and column components were modelled assuming lumped plasticity at member ends using FEMA beam FEMA column elements. Shear wall components were modelled through fiber sections by assigning constitutive material models to the fibers. Hognestad model (1951) was used for simulating concrete material and Yalchin model (2000) was used to represent steel material.

Incremental dynamic analysis (IDA) (Vamvatsikos and Cornell 2002) was applied to the structural model to estimate structural performance under different intensities of earthquakes. These analyses involve subjecting the structural model to a set of ground motion records, each scaled to multiple levels of intensity, resulting in curves of response

parameter versus intensity level. IDA involves selecting two parameters: seismic intensity measure and engineering demand parameter (damage indicator). Two different seismic intensity measures were considered consisting of; spectral acceleration of fundamental period with 5% of critical damping ( $Sa(T1),5\%$ ), and peak ground acceleration (PGA).  $Sa(T1)$  was used by previous researches including Vamvatsikos and Cornell (2002) and Ellingwood et al. (2007). This measure of intensity reflects both the characteristic of the earthquake and the structural period. It is defined in the National Building Code of Canada as a design parameter, and is frequently used by designers. On the other hand, scaling based on PGA gives a uniform format for the hazard, irrespective of the period of the structure, allowing comparison of the behaviour of buildings. Furthermore, scaling based on PGA is more straight forward and convenient to use. Inter-storey horizontal drift ratio was selected as the damage indicator. The use of inter-storey drift to define different limit states is quite common among engineers as it can be computed and rationalized easily.

Each set of 20 records selected was applied to the structure choosing ten scale factors to cover the structural behaviour from linear behaviour to collapse range. The results are demonstrated as IDA curves under western and eastern seismicity in Figure A2.

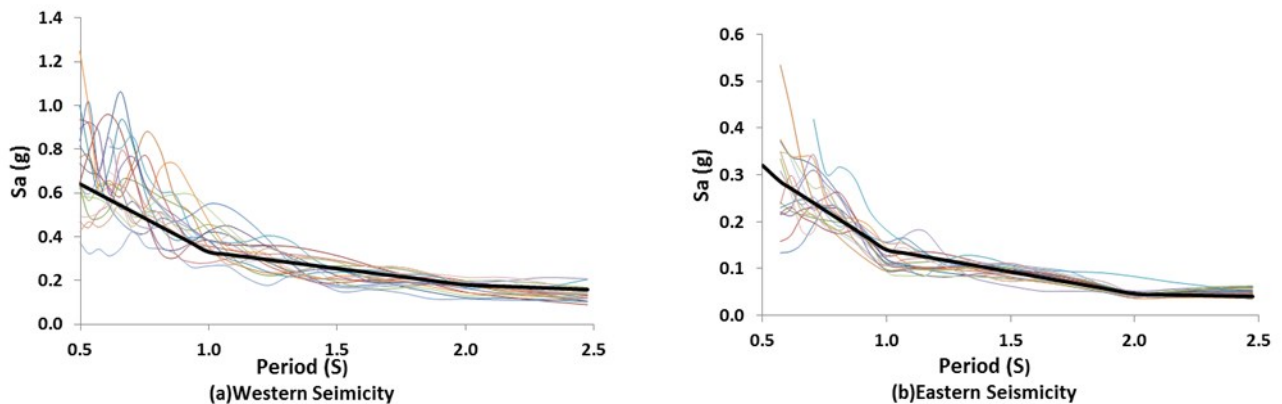


Figure A1: Comparison of UHS and spectral acceleration of selected records for Vancouver and Ottawa

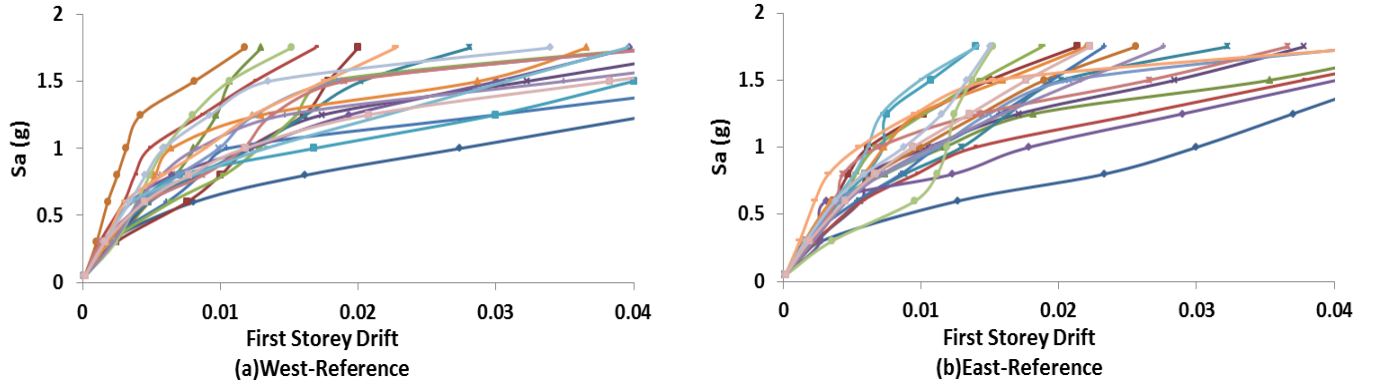


Figure A2: Result of IDA applied to reference building using western and eastern Canadian seismicity

## Fragility Analysis under Different Seismicity

IDA results provide the input for developing fragility curves as probabilistic tools representing the probability of exceeding predefined damage states under different levels of ground motion intensity. Fragility function is described in the form of Equation A1.

$$P[D > Di | IM] = \phi \left[ \frac{\ln(x / Di)}{\sqrt{\beta_{d/IM}^2 + \beta_c^2 + \beta_m^2}} \right] \quad \text{Eq. A1}$$

In the above expression,  $\phi(\cdot)$  is standard normal cumulative distribution function,  $Di$  is upper bound for each damage level,  $x$  is the median value of demand as a function of  $IM$ ,  $\beta_{d/IM}$  is the dispersion (logarithmic standard deviation) of demand conditioned on  $IM$ ,  $\beta_c$  is capacity uncertainty and  $\beta_m$  is modeling uncertainty.

Seismic fragility curves of the buildings are derived using the results of IDA, under both western and eastern seismicity conditions. Fragility curves depict probability of exceeding predefined damage states under different level of seismic intensity. Three

limit states of immediate occupancy, life safety and collapse prevention compatible with ASCE 41 definitions (ASCE 2007) were selected. More information on quantifying the limit states is available in authors' previous research.

As Figure A3 indicates, the comparison between the fragility considering western and eastern seismicity shows that the difference between fragilities for the two case fall in the range of 5% when  $S_a(T)$  used as the seismic intensity measure. One should be aware that the difference in seismicity dictates different probability of occurrence of each level of spectral acceleration for eastern and western site condition. Therefore this observation does not mean that the building is vulnerable to the same level whether it is located in Vancouver or Ottawa. In fact, comparing UHS for Vancouver and Ottawa indicates that the spectral acceleration of seismic event with 2475 years return period (2% probability in 50 years) for Vancouver is almost twice the value for Ottawa, dictating that the building has higher risks of failure if located in Vancouver rather than Ottawa. On the other hand, choosing PGA as the seismic intensity measure indicates remarkable difference for the same model under western and eastern seismicity as demonstrated in same Figure. The reason behind this observation is the difference in the nature of seismic events in western and eastern Canada. The generated records for Vancouver and Ottawa have different characteristic specifically longer duration of records generated for Vancouver compared to the ones generated for Ottawa. Having PGA as the seismic intensity measure reflects these differences, while having  $S_a(T)$  as the seismic intensity measure decreases the effects of site specific record selection.

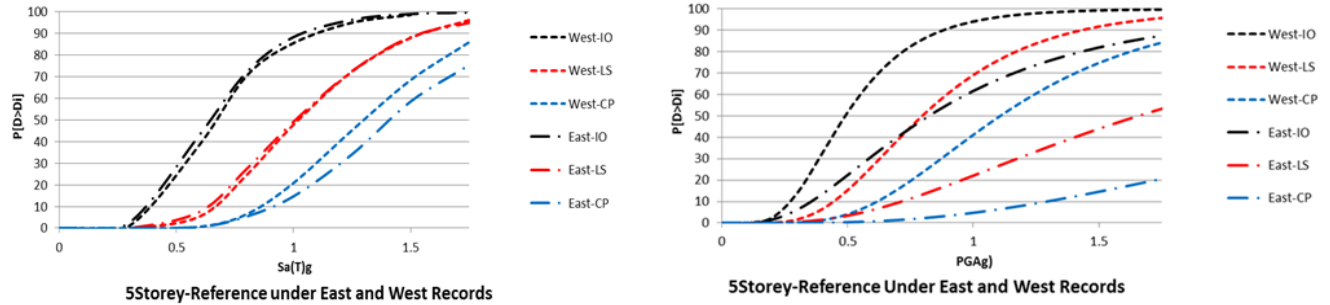


Figure A3: Comparison of behaviour of reference building subjected to seismicity of east and west

## Direction of Applied Load

In previous sections, time history records were applied in one direction. ASCE 7-10 standard on minimum design loads requires applying time history records in both horizontal directions in 3D analysis, while there is no such requirement in Canadian codes. In this study, the purpose was seismic assessment of building as opposed to design. Therefore, the effect of direction of applied load needed further investigation to find the detrimental condition. Hence, the reference structure was analyzed under time histories applied in X direction, and it was also analyzed under records at 45 degree direction. Figure A4 demonstrates the results of IDA while time histories are applied at 45 degree, and Figure A5 shows comparison of derived seismic fragility curves for reference building under applied records at different directions. Based on the results, when scale factors are smaller and the structure is essentially in linear behaviour range, applying the time history records in the X direction forces the structure to undergo higher drifts resulting in higher damage, compared to the case when the same load is applied in 45 degree. Applying the load at 45 degrees direction results in lower values of drift, as the load in each direction is about 70% of the load of the unidirectional case. Moreover, changing the behaviour to nonlinear range is delayed in such analyses for the same reason. However, in higher range of spectral acceleration especially near collapse, the

behaviour of the building under bidirectional load becomes worse and it leads to collapse at earlier values of drift. Therefore, as a general conclusion for shear wall buildings without torsional sensitivity, applying seismic load in one direction is accurate and safe enough.

## Discussion and Conclusion

Seismic fragility analysis of existing buildings assesses the building performance through probabilistic approach. In order to derive analytical fragility curves, detailed numerical simulation of the structure and numerous time history analyses are necessary. Therefore, parameters affecting the results require evaluation and some of them need to be eliminated before analyzing different building categories and performing time consuming analyses. Herein, the effect of site specific record selection for cities with western and eastern Canadian seismicity characteristics was assessed with both spectral acceleration and peak ground acceleration as the seismic intensity measures.

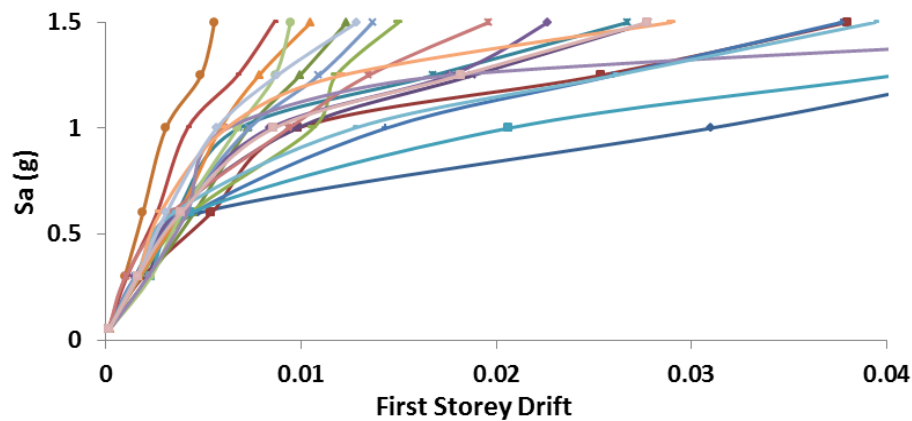


Figure A4: Result of IDA applied to reference building with seismic load applied in 45 degree

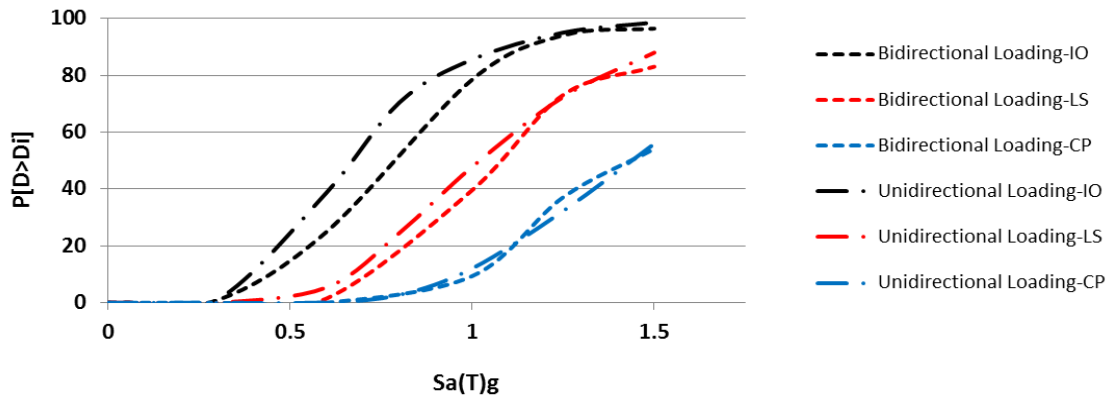


Figure A5: Comparison of fragility curves of reference shear wall building under time histories with different directions

The fragility curves derived for western and eastern seismicity based on spectral acceleration have similar shapes with a margin of 5% difference. This observation indicates that choosing spectral acceleration as the indicator of seismic hazard would give a normalized format of hazard and the results derived for one site can provide information for other sites having similar building design. However, this does not indicate similar seismic vulnerability of the building in western and eastern sites. To make a judgement on vulnerability of the building in each site, the fragility of the buildings under expected levels of seismic hazard for that site must be used. As an illustrative example, NBCC design spectral acceleration which is based on seismic hazard with 2% probability in 50 years for the 5-storey building located in Vancouver is twice the value for similar building in Ottawa.

When peak ground acceleration is chosen as seismic hazard intensity parameter, the difference in site specific records becomes more significant and obvious. Accordingly, at each PGA value the western time history records are indicators of an event with longer duration and possibly several ups and downs, while the eastern records represent events with significantly shorter durations enforcing less deformation on the structure.

The last investigation was performed on the direction of applied load with time history records applied parallel to one of the principal axes and at a 45 degree angle with respect to both planar axes. When the same records are applied at 45 degrees, each component has a scalar value equal to 0.7 times the load magnitude. As a result, application of the records at an angle initially imposed lower drift values. Moreover, the building nonlinear response was delayed in this case. However, at higher values of seismic hazard and especially for the collapse prevention limit state, more severe damage is possible when bidirectional records are applied with a margin of 5% difference in fragility results. The overall behaviour under unidirectional application of the seismic load was more severe and the conclusion is that for future investigation on fragility analysis, it is safe to apply the load parallel to the main axis.

## References

- ASCE. 2007. Seismic rehabilitation of existing buildings. ASCE 41 Reston, VA American Society of Civil Engineers.
- ASCE. 2010. Minimum design loads for buildings and other structures. American Society of Civil Engineers, Reston, Virginia.
- Atkinson, Gail M. 2009. "Earthquake time histories compatible with the 2005 National building code of Canada uniform hazard spectrum." *Canadian Journal of Civil Engineering* no. 36 (6):991-1000.
- Celik, Ozan Cem, and Bruce R Ellingwood. 2009. "Seismic risk assessment of gravity load designed reinforced concrete frames subjected to Mid-America ground motions." *Journal of structural engineering* no. 135 (4):414-424.
- CSI. 2013. Perform 3D Version 5.0.1. Nonlinear Analysis and Performance Assessment of 3D Structures. Berkeley, CA: Computers & Structures, Inc.
- Ellingwood, Bruce R, Ozan Cem Celik, and Kursat Kinali. 2007. "Fragility assessment of building structural systems in Mid-America." *Earthquake Engineering & Structural Dynamics* no. 36 (13):1935-1952.
- Hognestad, Eivind. 1951. "Study of combined bending and axial load in reinforced concrete members." University of Illinois. Engineering Experiment Station. Bulletin; no. 399.

- Koduru, SD, and T Haukaas. 2009. "Probabilistic seismic loss assessment of a Vancouver high-rise building." *Journal of structural engineering* no. 136 (3):235-245.
- NRCC. 1995. *National Building Code of Canada*. Ottawa, Canada: National Research Council of Canada.
- NRCC. 2010. *National Building Code of Canada*. Ottawa, Canada: National Research Council of Canada.
- Pejovic, Jelena, and Srdjan Jankovic. 2015. "Seismic fragility assessment for reinforced concrete high-rise buildings in Southern Euro-Mediterranean zone." *Bulletin of Earthquake Engineering*:1-28.
- Ramamoorthy, Sathish K, Paolo Gardoni, and Joseph M Bracci. 2006. "Probabilistic demand models and fragility curves for reinforced concrete frames." *Journal of structural Engineering* no. 132 (10):1563-1572.
- Vamvatsikos, Dimitrios, and C Allin Cornell. 2002. *The incremental dynamic analysis and its application to performance-based earthquake engineering*. Paper read at Proceedings of the 12th European Conference on Earthquake Engineering.
- Yalcin, C, and M Saatcioglu. 2000. "Inelastic analysis of reinforced concrete columns." *Computers & Structures* no. 77 (5):539-555.
- Zareian, Farzin, and Helmut Krawinkler. 2010. "Structural system parameter selection based on collapse potential of buildings in earthquakes." *Journal of structural engineering* no. 136 (8):933-943.

# **Appendix B: Seismic Assessment of a 12-storey Shear Wall Building Using the Fragility Curves Developed**

A 12-storey reinforced concrete shear wall building is selected as an illustrative example to demonstrate the application of the fragility curves developed. The building has been adopted from the seismic design Chapter of the Concrete Design Handbook (Cement Association of Canada, 2006). It was designed for Montreal on stiff soil. The plan and elevation layout of the building is shown in Figure B.1.

This structure is an office building with twelve storeys. The structural system has a centrally located elevator core and gravity frames. Each floor consists of a 200 mm tick flat plate with 6 m interior spans and 5.5 m end spans. The columns are 550x550 mm, and the thickness of the core wall is 400 mm. The design conformed to the ductile detailing requirements of CSA A23.3-2004.

The fundamental lateral period of the structure in both the N-S and E-W direction is 0.87 sec, computed based on the empirical expression given in the 2005 NBCC. The calculated period for the structure using ETABS is 1.83 sec in N-S direction and 1.72 sec

in E-W direction. The design period used was limited to twice the value of the empirical value, resulting in 1.74 sec.

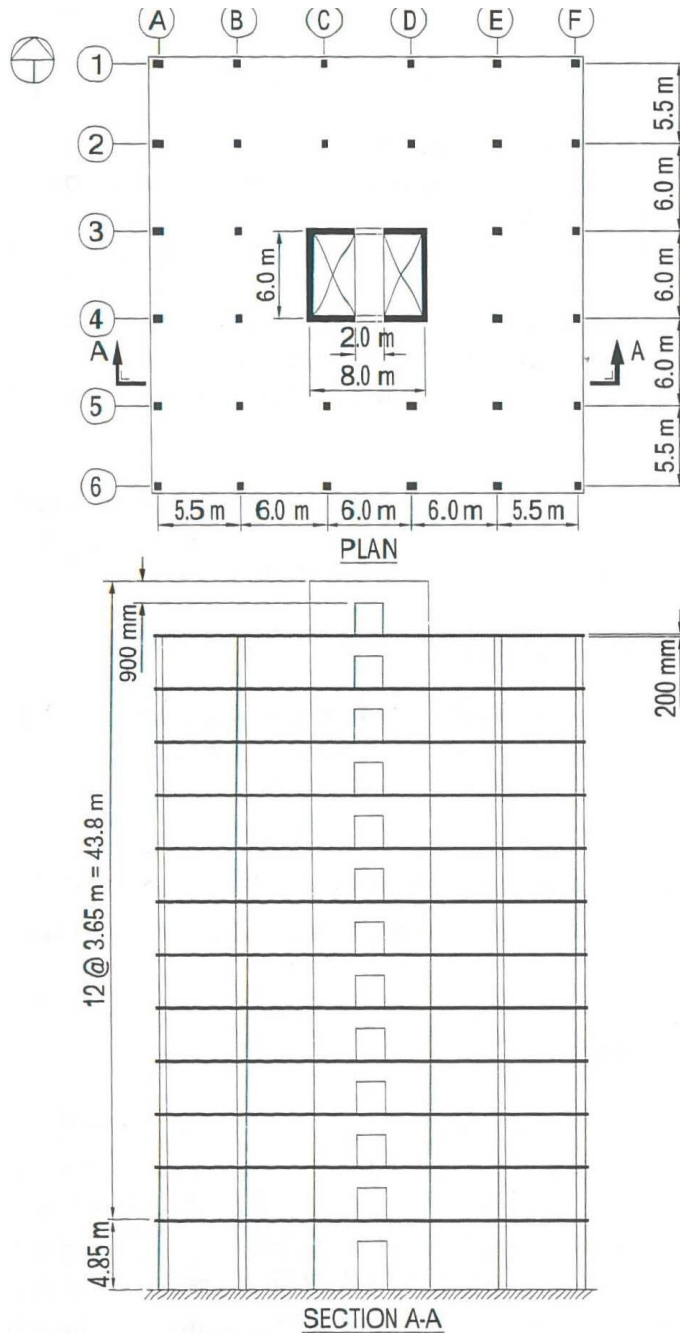


Figure B1: Plan and elevation views of selected 12-storey building

The building falls within the range of height and period as the 10-storey building considered in the research project (40 m height, code empirical period = 0.80 sec, first lateral period = 1.72 sec and design period = 1.60 sec). Also, Montreal and Ottawa had similar values of  $S_a$  for periods smaller than 2 sec in the 2005 NBCC. Therefore the fragility curves generated for Ottawa could be used for seismic assessment of the selected shear wall building.

The spectral acceleration for Montreal is  $S_a(T) = 0.07g$  for the maximum permissible dynamic period of 1.74 sec found for the building. Based on the stiff soil of the site (Type D), the velocity based foundation factor is determined to be  $F_v=1.36$ . This results in the design spectral acceleration of  $S(T) = 0.10g$ . Using the code empirical period, the value of spectral acceleration would be  $S_a(T) = 0.26g$  and  $S(T) = 0.30g$ . The fragility curves were generated using records intended for firm soils (reference soil condition for which the soil modification factor is 1.0). The effect of soil condition can be reflected approximately through the use of soil modification factors, as it is done in the NBCC equivalent static load approach. Therefore, for seismic assessment of the 12-storey building considered in this example, the spectral acceleration in the fragility curves correspond to  $S(T)$  that incorporates the effects of soil condition.

The example Montreal building was designed for full ductility ( $R_d = 3.5$ ). The fragility curves for Ottawa were developed for buildings designed to perform in a moderately ductile manner. However, at the NBCC design level earthquakes, the Ottawa buildings did not develop significant inelasticity. Hence, the additional deformability available in buildings due to moderately ductile or fully ductile designs was not utilized at code level earthquake intensity. Therefore, the use of the Ottawa fragility curves, given in Fig. B2, in assessing a building designed and detailed as a fully ductile building in Montreal is believed to yield acceptable results. Using the fragility curves for the 10-storey shear wall building in Ottawa and the empirical design period for the 12-storey Montreal building resulted in 50% probability of exceeding immediate occupancy, 15%

probability of exceeding life safety, and 2% probability of exceeding collapse prevention limit states. When the empirical period is increased by a factor of 2.0, as permitted by the 2005 NBCC because of the increased dynamic period computed, it was observed that the building had 3% probability of exceeding the immediate occupancy performance level. In this case the building had zero percent probability of exceeding the life safety performance level.

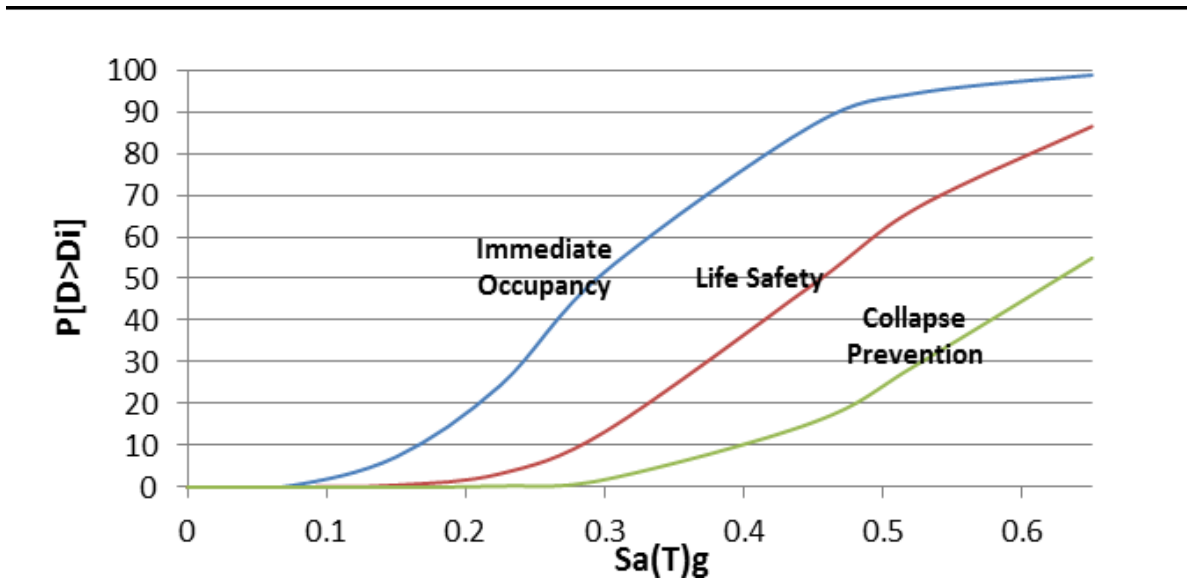


Figure 02: Fragility curves derived for 10-Storey shear wall building used for seismic assessment of 12-storey building in Montreal