Effects of Long Duration Ground Motions on the Probability of Drift Exceedance for Reinforced Concrete Frames near the Cascadia Subduction Zone

by

David Holl Ling Chin

B.A.Sc, The University of British Columbia, 2013

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF
THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF APPLIED SCIENCE

in

THE FACULTY OF GRADUATE AND POSTDOCTORAL STUDIES

(Civil Engineering)

THE UNIVERSITY OF BRITISH COLUMBIA

(Vancouver)

October 2017

Abstract

There are discrepancies in responses when a structure is subjected to short and long duration ground motions. The probability of drift exceedance has been repeated reported to increase under the influence of long duration records. Although the 2015 National Building Code of Canada has probabilistically taken the Cascadia Subduction Zone seismic hazards into considerations, none is known as what might happen to structures designed using this new Code if subjected to a large magnitude subduction earthquake.

The answer is found via computer simulation. Following the general approach adopted in the Seismic Retrofit Guidelines 2nd Edition, incremental dynamic analysis is conducted to investigate discrepancies in the probability of drift exceedance for certain building types under both crustal and subduction ground motion records. These ground motions are selected and scaled to match the 2015 uniform hazard spectrum of Victoria, B.C. A simple shear wall model is first examined to generalize the effects of long duration ground motions. Then a similar study on a reinforced concrete frame is conducted to confirm these generalizations.

Long duration ground motions seem to cause a higher probability of drift exceedance in moderately ductile buildings. However, no effect is observed in non-ductile and highly ductile buildings. The system internal capability to dissipate seismic energy by means of hysteretic loops is also contributing to the overall probability of drift exceedance. Its effect is more evident in the long duration as there are more load reversal cycles. Results discussion is provided, and potential ways to account for long duration in structural design are recommended.

Lay Summary

The key goal of the research is to make buildings stronger in a seismic event. Based on observations from past earthquakes, deficiencies in the current building code have been identified. This research contributes to the efforts in resolving these deficiencies in three steps. Step one is to develop computer building models to understand the effects of the deficiencies. Step two is to analyze the root cause why the observed effects are inevitable. Step three is to come up with recommendations to tackle the root cause so as to mitigate the observed adverse effects due to the deficiencies. Outcome of this research will help to promote safer structures in everyone's community.

Preface

Ground motion information presented in Chapter 2 and 3 are based on work conducted at UBC's Earthquake Engineer Research Facility (EERF) by Dr. Armin Bebamzadeh and Mr. Michael Fairhurst. I was responsible for compiling the information and presenting at one of the structural seminars held weekly at the Civil Engineering and Mechanical Engineering Building (CEME).

The methodology in Chapter 3 and study results in Chapter 4 and 5 of this thesis were partially published and presented at the 11th Canadian Conference on Earthquake Engineering (Victoria, British Columbia, 2015). The paper can be found in the conference proceedings in the title: Paper 94011, "Effects of Subduction Ground Motions on the Probability of Collapse on Low-Rise Buildings".

The computer model used in Chapter 5 was generously shared with my research team by Abbie Liel of University of Colorado Boulder. I have developed and refined the building simulation models to fit the purposes in this thesis. I have also refined a series of data post-processing programming scripts to generate the necessary diagrams and illustrations. Chapter 6 is my in-depth analysis and discussion of the study results under direct supervision from Dr. Carlos Ventura.

Dr. Carlos Ventura, Dr. Armin Bebamzadeh and Mr. Michael Fairhurst have helped to edit and proofread this manuscript, and have provided me with valuable guidance and engaging discussion throughout the course of my study.

Table of Contents

List of Tables

List of Figures

Acknowledgements

I offer my enduring gratitude to the faculty, staff and my fellow students at the UBC, who have inspired me to continue my work in this field.

I owe particular thanks to Dr. Carlos Ventura whose enlightening ideas have taught me to seek an answer more thoroughly. I am truly thankful for your sincere help when I applied for the grant support because without it, I could not have gone so far.

I thank Dr. Armin Bebamzadeh for enlarging my vision of research in earthquake engineering. It has been a very engaging lesson to witness the paper submission process. My sincere congratulations to your award received for the hard work done on the Seismic Retrofit Guidelines. I must also express my gratitude here for the kind resource sharing opportunity facilitated by Dr. Bebamzadeh with Mrs. Abbie Liel of University of Colorado Boulder.

Special thanks are owed to Mr. Michael Fairhurst, who has always been a helpful colleague and resourceful friend and is never tired of my endless questions on programming and script writing. I often find your enthusiasm encouraging and inspiring in our small research office.

Dedicated to RR and CW…

Chapter 1: Introduction

1.1 Problem Statement

Ground motion duration is not explicitly considered in modern building codes. By studying historical subduction earthquakes elsewhere in the world, the earthquake engineering community in Canada is aware of the long and intense ground shaking resulting from the sudden eruption of a subducting tectonic plate. Since the West Coast of Canada lies just kilometres from the Cascadia Subduction Zone, long duration ground motion hazard is real and must be accounted for.

The goal of this research is to identify possible ways to mitigate the risk of building collapse in long duration ground shaking. Theoretical models are studied to gain some general insights to what may happen to a structure when subjected to short and long duration ground motions. Results in this research shall pave the way towards future building code development for a ground motion duration design procedure.

1.2 Summary of Research Outcomes

Ground motion duration matters to structural design. The state-of-the-art force based approach in the current building code has its shortcomings. As demonstrated later in this thesis, a code-satisfying building will exhibit a larger probability of drift exceedance when subjected to long duration ground motions than to the short duration counterparts, even though both types of ground motion are in compliance with the prescribed seismic hazards.

Performance based design is a useful procedure when dealing with long duration ground motion. By introducing nonlinearity in the structural design, the engineers are able to control the extent of deformation and thus, meeting the life safety criterion. Two parameters seem to dominate performance based design: ductility and hysteretic behaviour.

The probability of drift exceedance is largely reduced when ductility is introduced to the structural system. This observation holds true under both short and long duration ground motions. Nonetheless, introducing ductility to an already ductile structure has almost no effect on the probability of drift exceedance. Retrofitting strategies must therefore require a good estimate of the structure's ductility in order to effectively combat long duration ground motions.

The hysteretic behaviour of the structural members also plays a role in the probability of drift exceedance of a structure. Although not as sensitive as ductility, the degree of material deterioration defined in the hysteretic model does affect the structure's ability to dissipate the input seismic energy. Caution must therefore be exercised when modeling hysteretic behaviour within the structural components.

1.3 Literature Review

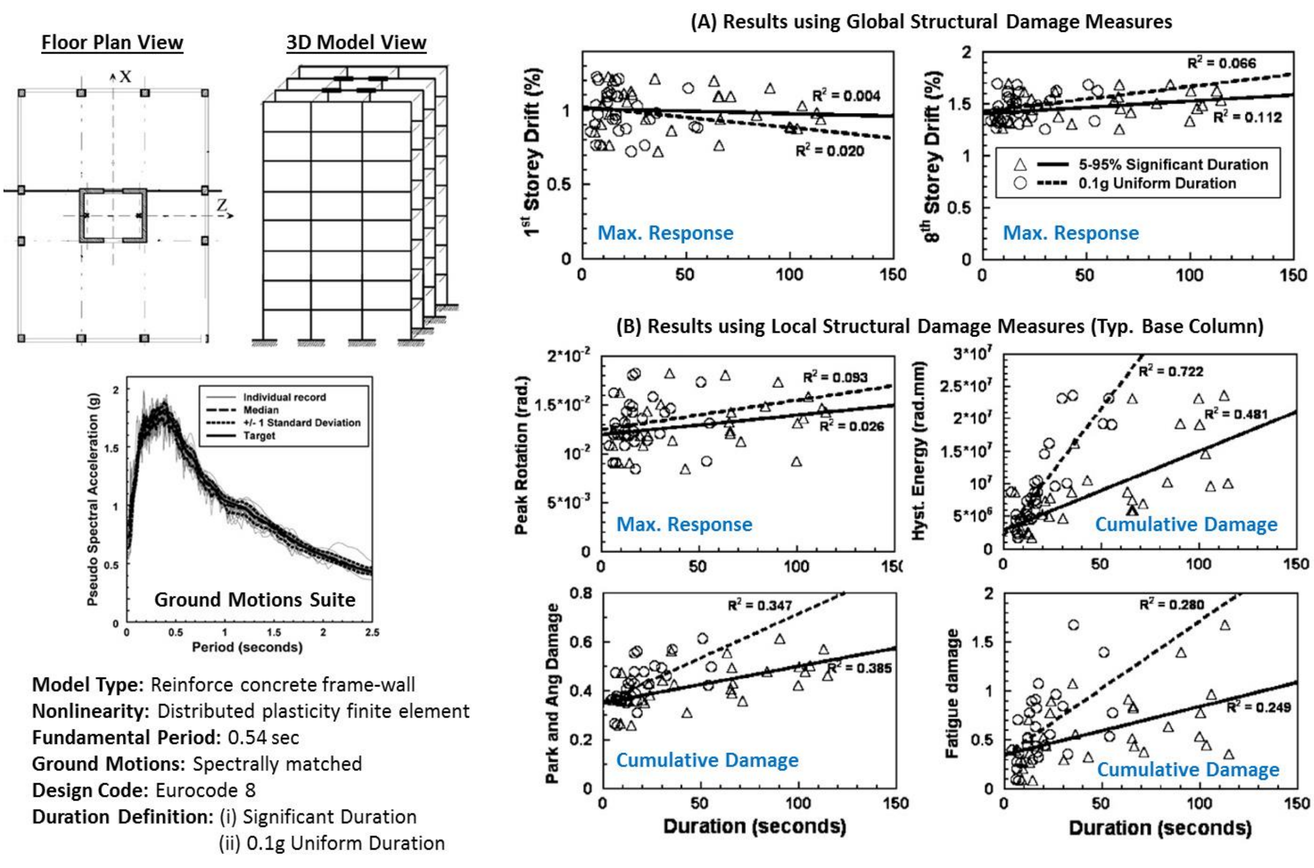
The effect of strong ground motion duration remains inconclusive to many researchers. Recent studies and relevant information are provided in a chronological manner to show the efforts leading to the current state of knowledge.

1.3.1 Hancock, J., and Bommer, J. J. (2006)

This paper establishes the general view on the influence of strong ground motion duration and structural damage. By reviewing a large amount of studies conducted before and during the millennium, Hancock and Bommer conclude: (i) that duration is positively related to structural damage when cumulative damage measures, such as the Miner's Rule for fatigue accumulation, are used; and (ii) that duration is poorly co-related to structural damage when maximum structural responses, such as displacement or drift, are used. The authors have also published results in agreement with this conclusion in the subsequent year (Hancock and Bommer, 2007). Figure 1 summarizes this subsequent study.

Recommendations to tackle the few shortcomings in these past studies are also provided. Since the failure mode differs for each structural component, duration alone is not sufficient for estimating structural damage. Hancock and Bommer suggest that future studies should specify the type of structure under investigation, the primary structural design parameters, and the metric used to quantify damage and duration, so as to draw insightful conclusions.

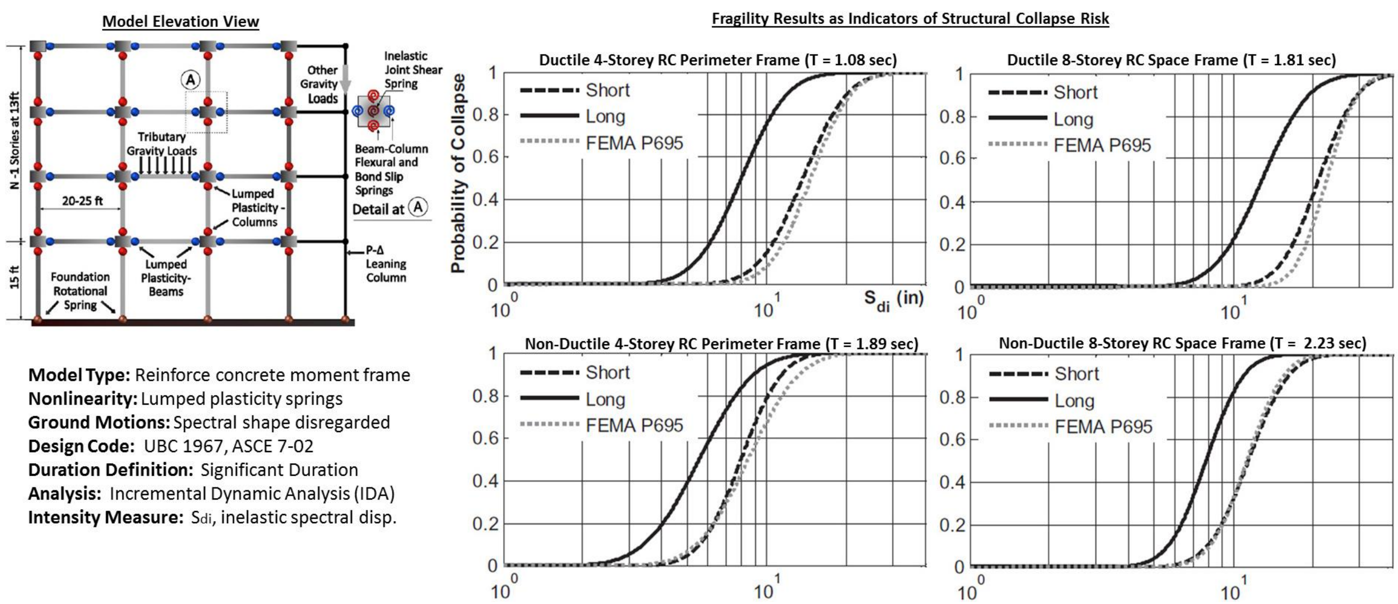
Figure 1 – Summary of duration and structural damage correlation study by (Hancock and Bommer, 2007)



1.3.2 Raghunandan, M. and Liel, A.B. (2013)

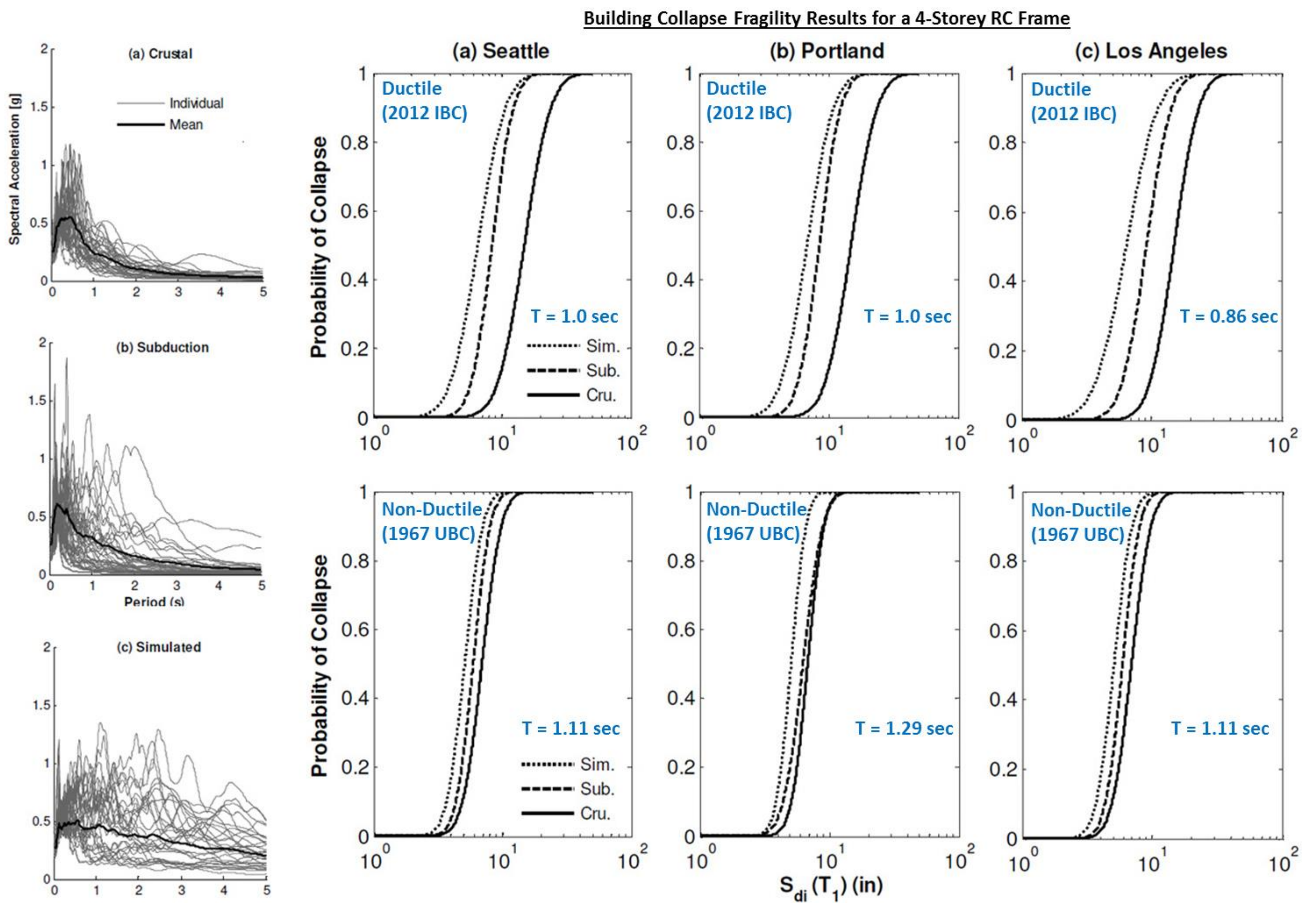
This paper investigates the influence of ground motion duration on structural collapse risk. Through computer simulation and statistical inference, Raghunandan and Liel have found agreement with Hancock and Bommer (2006), and have demonstrated that as the duration increases, the risk of collapse also increases due to higher energy demands. Their results are shown in Figure 2. They advise an informative collapse risk assessment should reflect ground motion intensity, frequency content, and duration through a careful selection of ground motions and choices of collapse risk indicators.

Figure 2 – Summary of structural collapse risk fragility results concerning ground motion duration by (Raghunandan and Liel, 2013)



The influence of duration is subsequently quantified in Raghunandan’s doctoral dissertation (2013), and refined in a follow-up publication (Raghunandan, M., et. al., 2015). Raghunandan has demonstrated that longer duration ground motion records induced by subduction earthquake will pose higher structural collapse risk than the shorter records induced by crustal earthquake. There are three key conclusions: (i) the influence of duration is more evident on ductile buildings than non-ductile ones; (ii) the risk of collapse of older reinforced concrete frame building is about seven times larger than that of modern ductile frames; and (iii) application of some adjusting factors to artificially increase the seismic design intensity seems to be a plausible way to account for collapse risk from subduction earthquakes when conducting seismic performance assessment. The fragility results are shown in Figure 3.

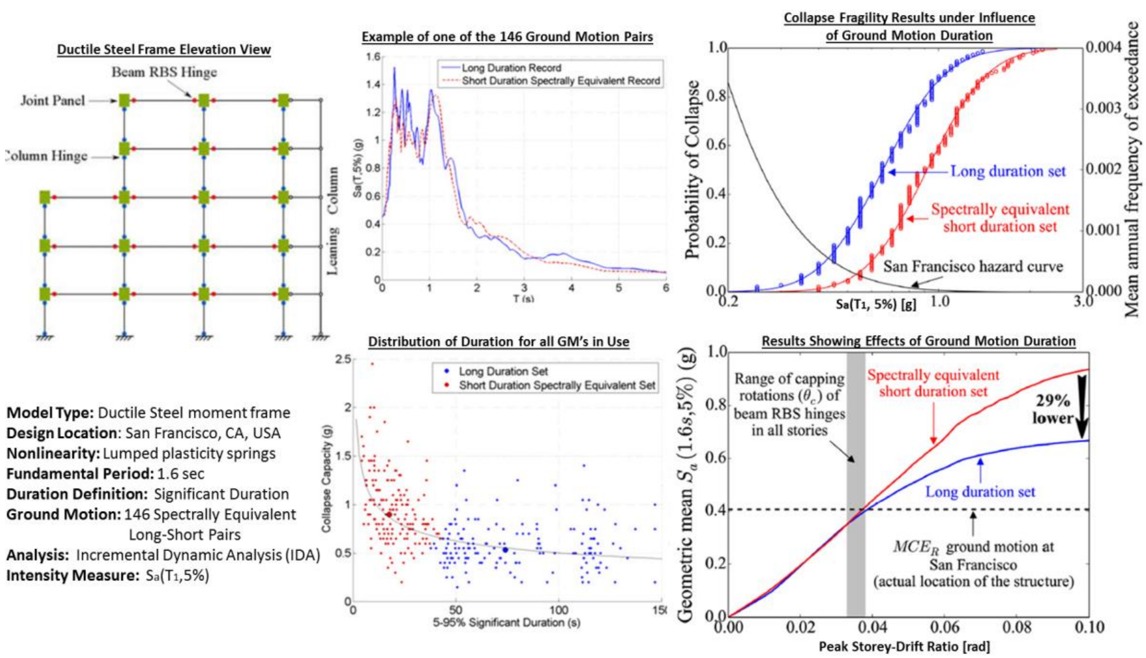
Figure 3 – Summary of frame building fragility results concerning seismic hazards at site and ground motion duration by (Raghunandan, 2013)



1.3.3 Chandramohan, R., Baker, J.W., and Deierlein, G.G. (2014a)

The research team at Stanford University examines the influence of ground motion duration on the collapse capacity on a ductile steel moment frame and a reinforced concrete bridge pier. Using 146 sets of spectrally equivalent long and short ground motion duration pairs, the team has found a 29% and 17% drop in the estimated median collapse capacity when using the long duration set, for the steel frame and bridge pier respectively. Sensitivity analyses also indicate that structures with high deformation capacity and rapid rate of cyclic deterioration are most influenced by ground motion duration. In order to maintain a satisfactory margin of safety against collapse, inferences were made to the results shown in Figure 4, that (i) non-linear analyses should be conducted at higher ground motion intensities to capture the influence of duration, and that (ii) adjustment factors for design strength and/or ductility requirements may perhaps be applied to geographical locations where long duration ground motions are likely to occur.

Figure 4 – Summary of collapse fragility results of a ductile steel frame under the influence of ground motion duration (Chandramohan, R., Baker, J.W., and Deierlein, G.G., 2014a)



1.3.4 Chandramohan, R., Baker, J.W., and Deierlein, G.G. (2016)

This paper is perhaps the first to explicitly account for ground motion duration in the design and assessment of structures. A proposed procedure to estimate the distribution of ground motion duration at a site is used in conjunction with source-specific conditional spectra to establish target spectra to which hazard-consistent ground motions are selected and matched. Figure 5 illustrates the set of ground motions that match both criteria. Multiple strip analysis is conducted to estimate the mean annual frequency of collapse, $\lambda_{collapse}$, of a ductile 8-storey reinforced concrete frame building.

The effect of ground motion duration is evident when the ground motion selection process is altered. If only the target spectrum is matched, or equivalently if duration is not considered, $\lambda_{collapse}$ will be underestimated especially for sites subjected to subduction earthquake hazards; whereas if the ground motions are selected based only on causal parameters, a generally accepted approach in current design codes, it will be overestimated. Figure 6 shows the fragility results for an 8-storey ductile reinforced concrete frame under all these three circumstances.

The study suggests further that to capture the effect of ground motion duration in code-based designs, collapse risk analysis results should be assessed and calibrated such that a factor may be applied to the code-based intensity level. This suggestion comes from the observation that lower intensity levels do not cause much deformation and deterioration in the nonlinear model, and thus unlikely to detect any influence of duration. This observation is consistent with the steel frame study by Chandramohan, R., et. al. (2015).

Figure 5 – The "*CS and Duration*" Ground Motion Set by (Chandramohan, R., et. al., 2016)

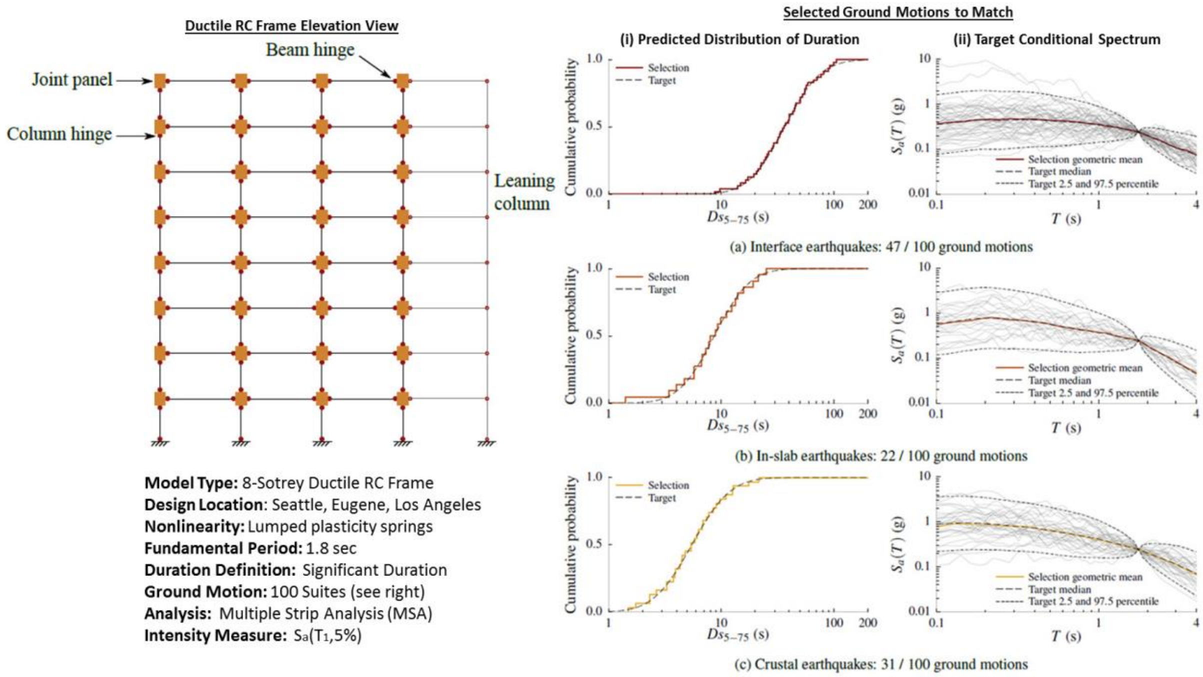
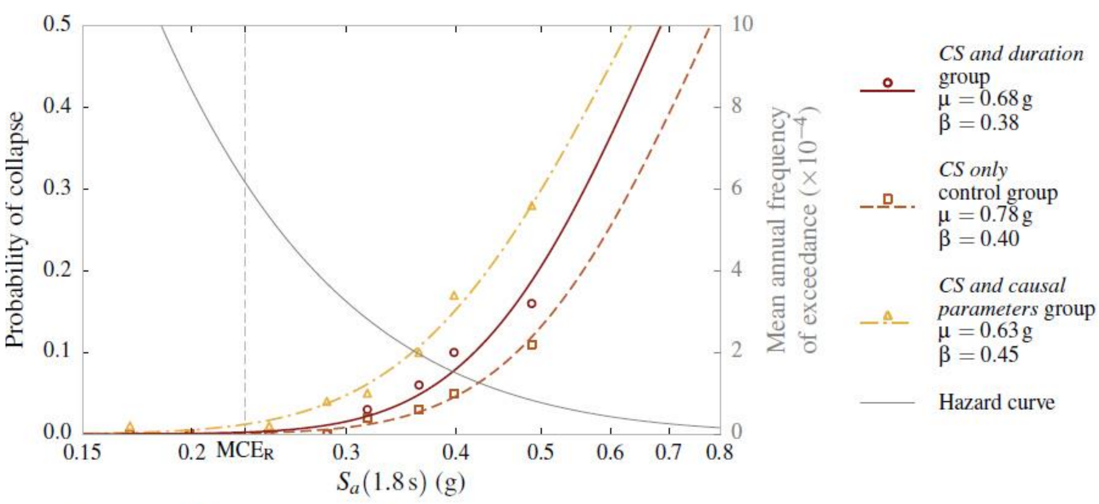


Figure 6 – Fragility Results of RC Frame by (Chandramohan, R., et. al., 2016)



Mean annual frequency of collapse ($\lambda_{collapse}$) of the reinforced concrete moment frame building.

Ground motion group	Seattle	Eugene	San Francisco
CS and duration group	5.4×10^{-5}	7.2×10^{-5}	15×10^{-5}
CS only control group	3.8×10^{-5} (−29 %)	2.9×10^{-5} (−59 %)	14×10^{-5} (−7 %)
CS and causal parameters group	8.2×10^{-5} (+53 %)	9.7×10^{-5} (+34 %)	23×10^{-5} (+51 %)

1.4 The Knowledge Gap

Analyses in the aforementioned literatures are rather time consuming and computationally demanding. There is yet to be a robust and efficient procedure to reasonably capture the effects of long duration ground motion without too much compensation on academic correctness. Summarizing from the Literature Review and inferring from the results of this thesis, the following might be a few feasible ways to explicitly account for long duration ground motions:

a. ground motion selection process

   As suggested by Chandramohan, R., et. al. (2016), a target duration spread can be established when selecting ground motions for analysis. This target duration spread can be predicted using already available tools. However, this process is rather time consuming.

b. restricting building types based on seismic region

   For example, unreinforced masonry construction should not be allowed in a seismic prone region. Engineers who wish to pursue the performance based design must demonstrate adequate ductility in the system, and that when the system is subjected to long duration ground motion, the probability of drift exceedance remains within a prescribe limit. This process would require a full update on the building code and is beyond the scope of this study.

c. protecting against deterioration

   For example, installation of dampers can effectively reduce seismic responses. However, damper design is not the same as accounting for long duration; hence this option is not considered.

d. increasing the seismic demand

In the force based design approach, a new long duration may be introduced to artificially increase the seismic demands so that a structure is designed to sustain larger loads, hence a larger cross section to deteriorate under a long duration ground motion. This approach is deemed rather reasonable and is computationally quicker than the other options. The primary focus of this thesis is to shed light on how to quantity such seismic demand increasing factor.

1.5 Scope of Study

Effect of long duration ground motion on reinforced concrete structures is the focus of the study. Existing tools, models, equations and procedures are utilized as fit. The design solution to long duration effects shall be as close to the code base approach as possible. Probability of collapse in terms of interstorey drift exceedance is the key indicator of acceptance in this study. It is evident from this study that the probability of drift exceedance will increase as the duration of ground motion increases.

Chapter 2: Cascadia Subduction Zone and Ground Motion Duration

2.1 Introduction

Seismic activities in the Cascadia Subduction Zone have long been recognized. Local legends, soil profiles, and tsunami records all point to a huge subduction earthquake that has occurred in the 1700s. Seismologists are anticipating another magnitude 9.0 earthquake to hit the West Coast of Canada anytime. Despite the inherent uncertainties in earthquake prediction, seismologists have resolved to adopt a statistical approach to estimate the earthquake magnitude by means of attenuation equations. Using these equations, a seismic hazard curve can be generated for a particular location. These hazards are then manipulated to generate a seismic force for structural analysis and design. In Canada, these steps are the state-of-the-art seismic design method, known as the Equivalent Static Force approach.

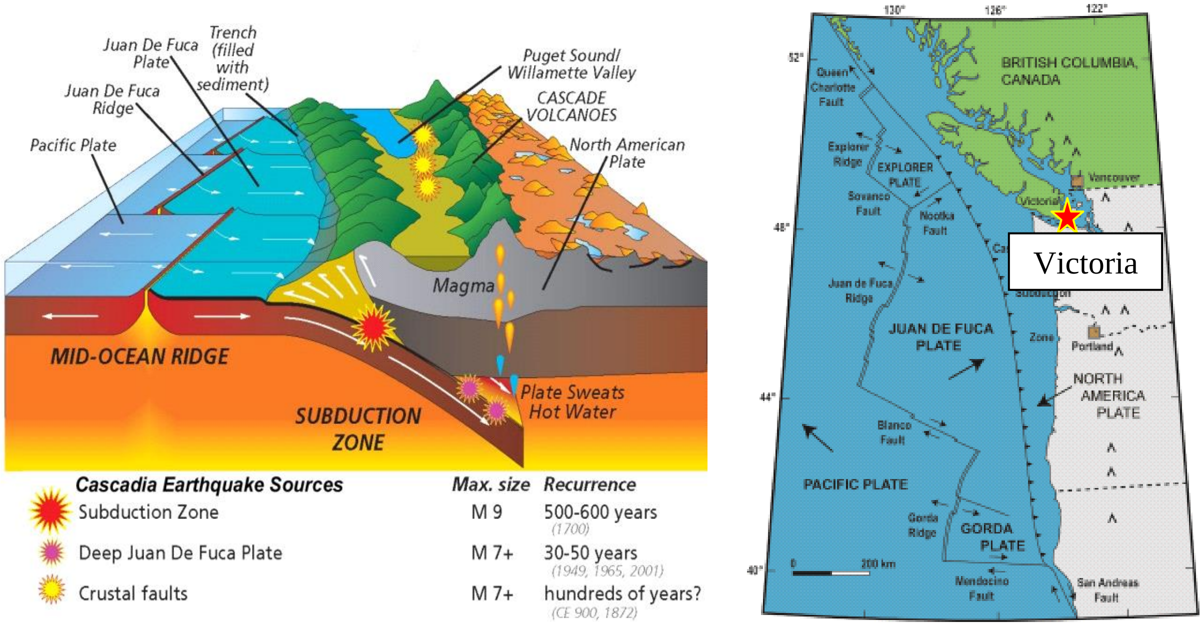
Seismic hazards due to the Cascadia Subduction Zone are now recognized in the 2015 National Building Code of Canada (NBCC 2015). Although the Equivalent Static Force approach can capture subduction ground motion hazards, the effect of duration is not explicitly considered. The goal of this research is to find out if the duration of ground motion has any influence on the structural collapse risk when a theoretical building is designed for the seismic hazards in NBCC 2015. The City of Victoria is selected for this research because of its economic significance to the Province of British Columbia and its geographical location in the Cascadia Subduction Zone.

2.2 Cascadia Subduction Zone

The West Coast of Canada is very close to the Cascadia Subduction Zone. The plan and section view of the Southwestern British Columbia region are shown in Figure 7. The heavier oceanic plates, Explorer and Juan de Fuca, slowly slide beneath the lighter crustal

North America Plate, creating a zone of high seismic activities. There are three types of earthquake that can occur in this region: crustal, sub-crustal, and subduction. Crustal earthquakes usually occur at shallow depth of above 20 km within the North America Plate. Sub-crustal earthquakes occur at a much deeper depth of more than 60 km below surface, typically within the subducting plates. Subduction earthquakes are rare event that can produce large devastating shaking, and are expected to occur at the interface between the Juan de Fuca and the North America Plate. The seismic hazards provided in NBCC 2015 have accounted for all three types of earthquake.

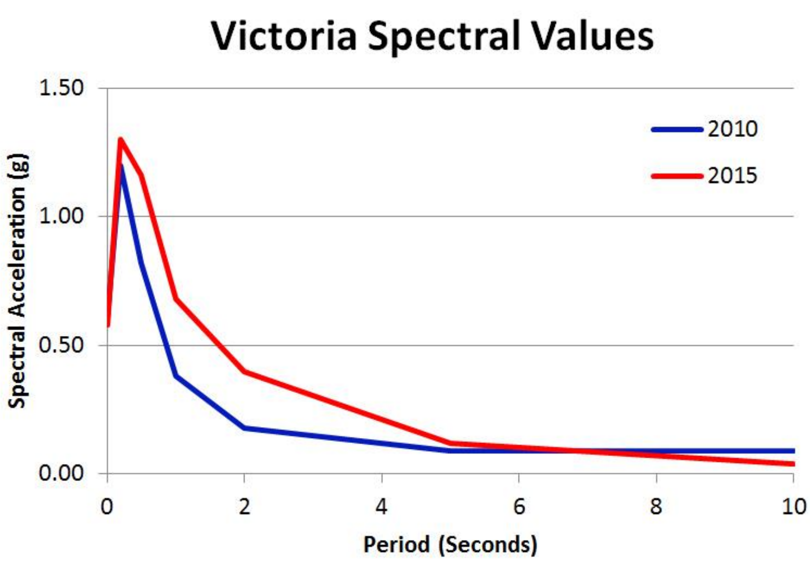
Figure 7 – The Cascadia Subduction Zone (U.S. National Park Service, 2014)



2.3 Seismic Hazards

The seismic hazard at a site is an estimation of how strong the ground may shake at the given location. It is often quantified as the lateral acceleration of the ground, expressed as some percentage of the gravitational acceleration. Using geological evidence and historical seismic records, seismologists have come up with models and procedures to compute the best estimate of the seismic hazards. These models are often referred to as attenuation equations, and the procedure to convert the model outputs into meaningful seismic hazard data is known as probabilistic seismic hazard analysis. Ultimately, seismic hazards at a location are represented by a hazard curve. Such curve depicts the fundamental period of a structure and the associating horizontal ground acceleration as a spectrum. To better account for seismic hazards arise from the Cascadia Subduction Zone, the NBCC 2015 committee have replaced the deterministic model used in NBCC 2010 by an updated probabilistic model, leading to the hazard difference shown in Figure 8.

Figure 8 – Comparison of Seismic Hazard Values at Victoria, B.C.



Period	2010	2015
PGA	0.61	0.58
Sa(0.2)	1.20	1.30
Sa(0.5)	0.82	1.16
Sa(1.0)	0.38	0.68
Sa(2.0)	0.18	0.40
Sa(5.0)	0.09	0.12
Sa(10.0)	0.09	0.04

One key observation from Figure 8 is the higher spectral values between periods of 0.2 s to 5.0 s. The increase in spectral values mainly comes from past experiences in the infamous Tohoku Earthquake that has caused tremendous devastation in Japan in March, 2011. The updated attenuation equations were developed using ground motions recorded during this event. Since the Cascadia Subduction Zone is somewhat tectonically similar, such devastating effects must be captured. Nonetheless, one updated hazard curve alone does not fully represent the long duration nature of the ground motions generated during the Tohoku subduction earthquake. It is the intent of this research to find out whether a structural design using the Equivalent Static Force approach with these new NBCC 2015 hazards is indeed acceptable.

2.4 Spectral Values and Ground Motion Duration

A seismic hazard curve does not fully capture the duration nature of a ground motion. Different researchers have proposed various measures of ground motion duration suitable to their respective studies. The concept of duration is first explored in this section. Following which is a summary of available duration measures. Advantages of the chosen “significant duration” are also discussed.

Duration of a ground motion is a record of the length of the shaking event. Two ground motions mays share the same peak ground acceleration but their duration can be completely different, as shown in Figure 9. Although the response spectrum is an effective tool to capture the maximum force effect, it does not reflect the duration aspect of the ground motion. It is therefore necessary to define duration using other tools.

Most available ground motion duration measures are derived from the acceleration time history records. To select the most suitable duration measure for this research, two hundred ground motions are selected based on a target spectrum and are applied to a hypothetic single degree-of-freedom (SDOF) model with varying ductility, see Figure 10. The scaling level of a ground motion to cause the top node to reach the collapse drift is recorded, and is then plotted against the duration measure of interest.

Figure 9 – Plot of Response Spectrum and Time History Accelerogram for two Ground Motions Sharing the Same Spectral Shape and Similar Peak Ground Acceleration (Chandramohan, R., Baker, J.W., and Deierlein, G.G., 2014a)

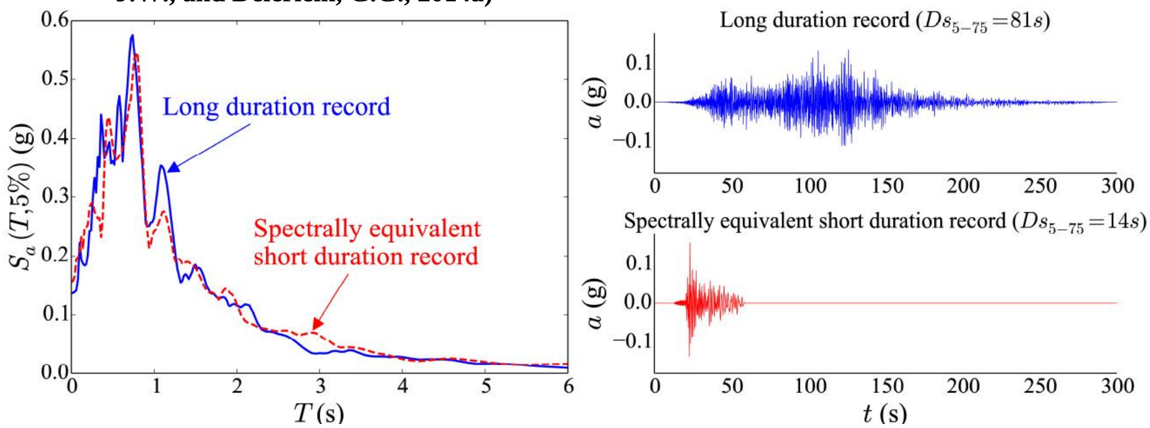
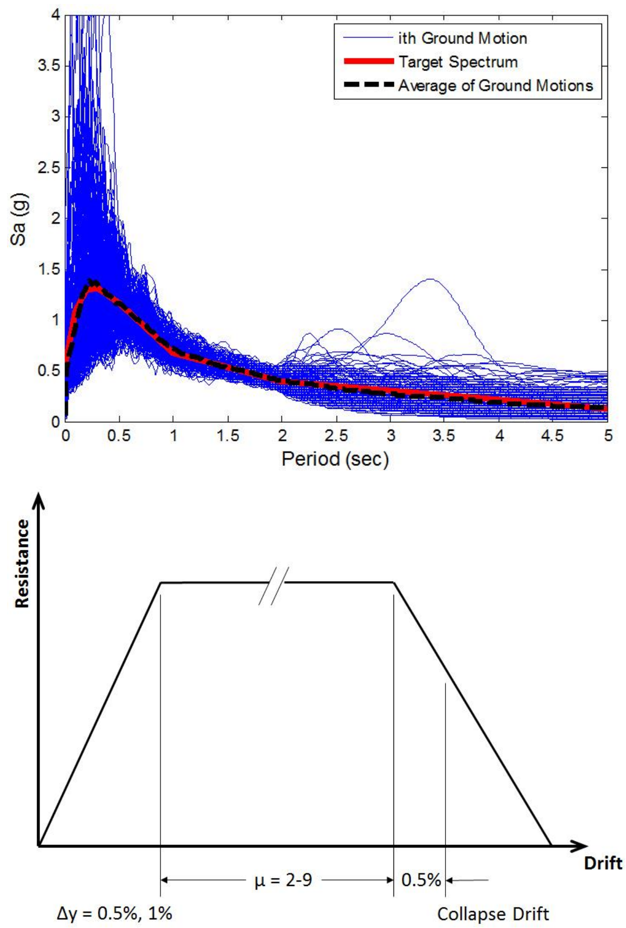


Figure 10 – (a) Selected 200 Ground Motion for Duration Measure Study (b) Backbone Curve for SDOF Model used in Duration Measure Study

(a)

(b)

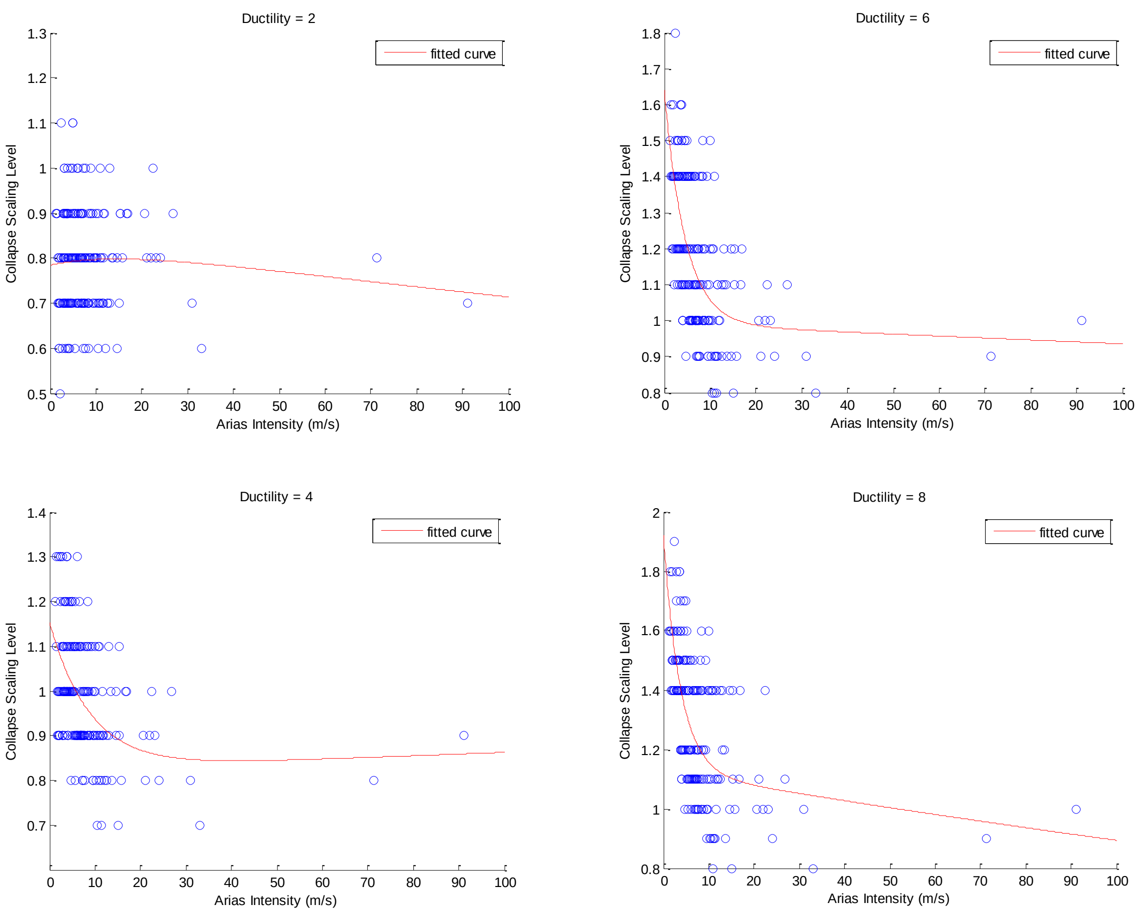


2.4.1 Arias Intensity

The Arial Intensity is defined as seen in Equation 2.4.1. In this equation, a(t) is the acceleration time history recorded during the ground shaking event and g is the gravitational acceleration. The results are shown in Figure 11.

$$I_a = \frac{\pi}{2g}\int_0^T a^2(t)dt \qquad \text{Equation 2.4.1}$$

Figure 11 – Collapse Scaling Level vs. Arial Intensity as Duration Measure

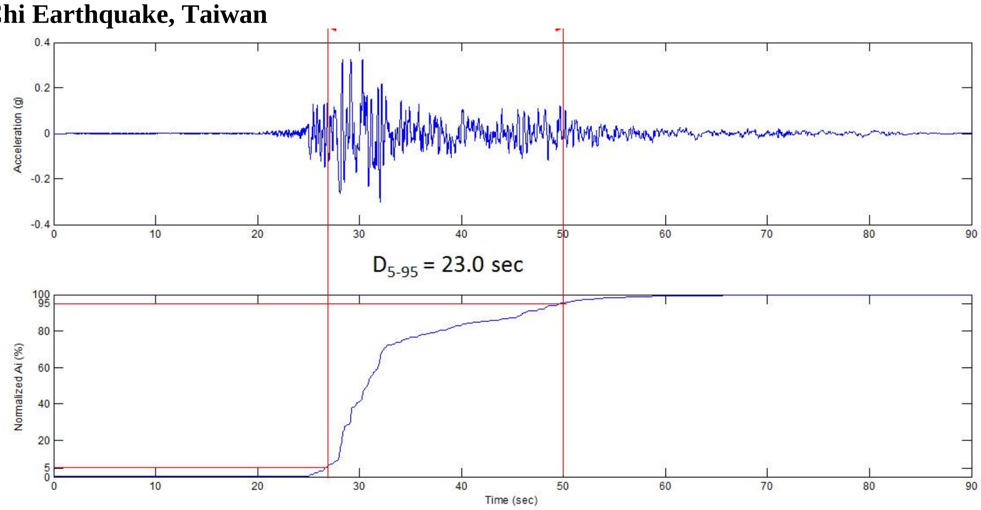


2.4.2 Significant Duration

Significant duration is similar to Arias Intensity. The 5-95% significant duration, denoted as $D_{5\text{-}95}$, represents the time it takes to accumulate 5% to 95% of the normalized area under the Arias Intensity integral. The significant duration is superior to Arias Intensity because the time history record can be in theory, infinitely long. The significant duration definition effectively filters out the portion of the time history record associating with the most vibrant shaking, as seen in Figure 12. Study results are shown in Figure 13.

Figure 12 – Illustration of the Definition of Arias Intensity and Significant Duration

(a) Chi-Chi Earthquake, Taiwan



(b) Tohoku Earthquake, Japan

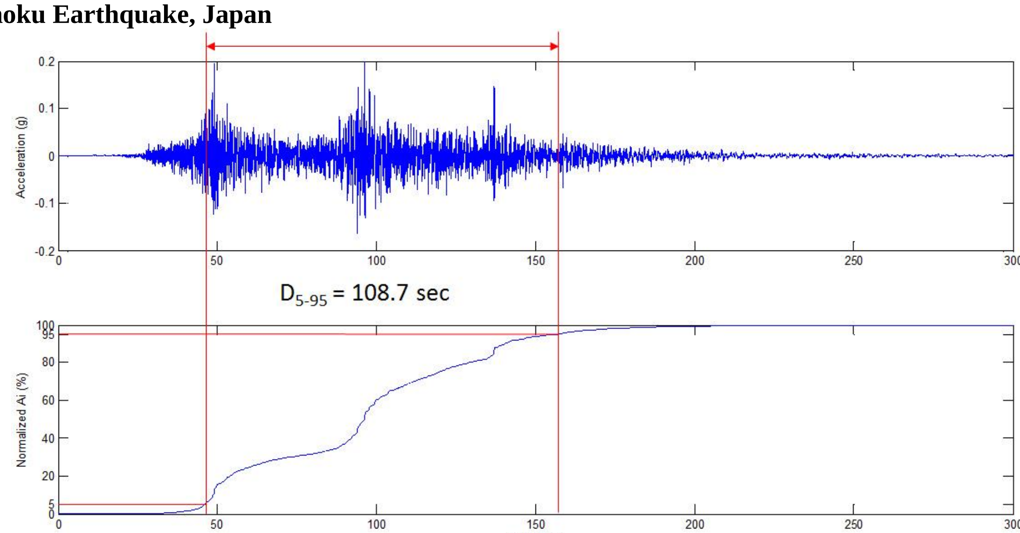
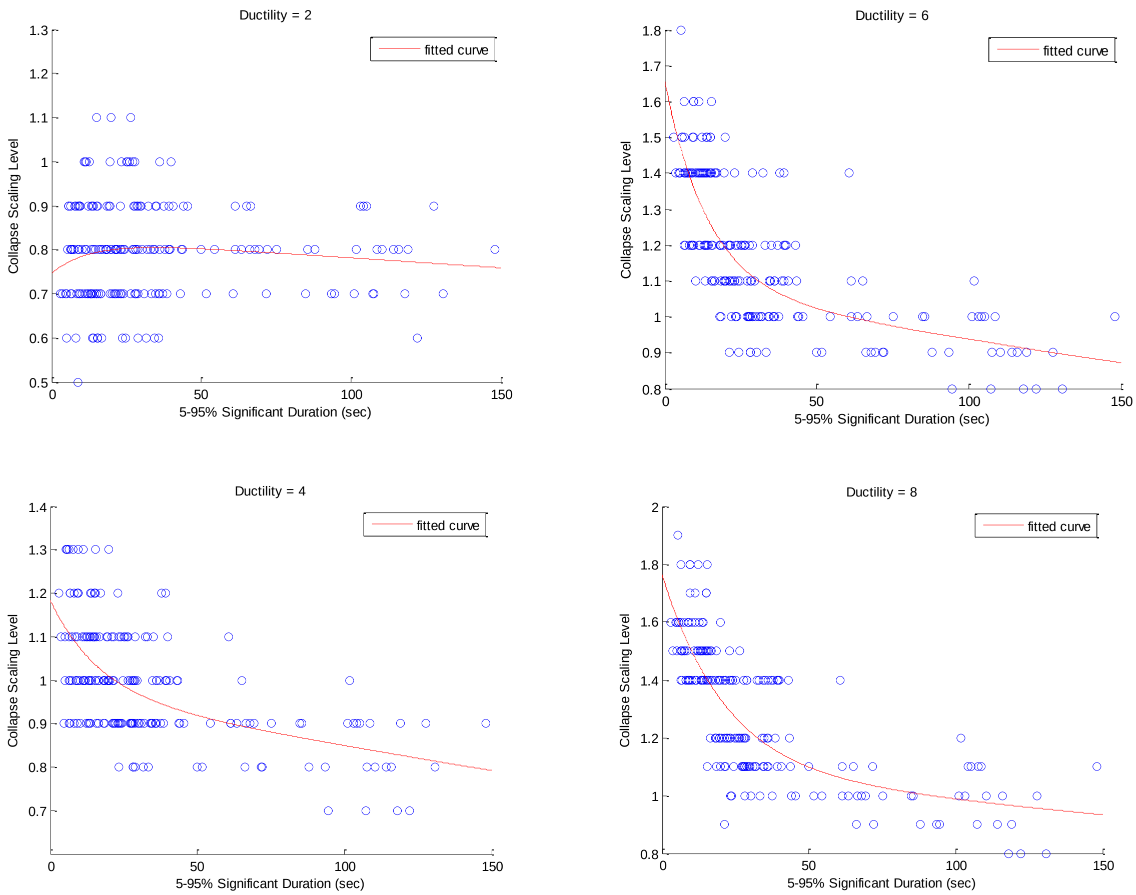


Figure 13 – Collapse Scaling Level vs. Significant Duration as Duration Measure

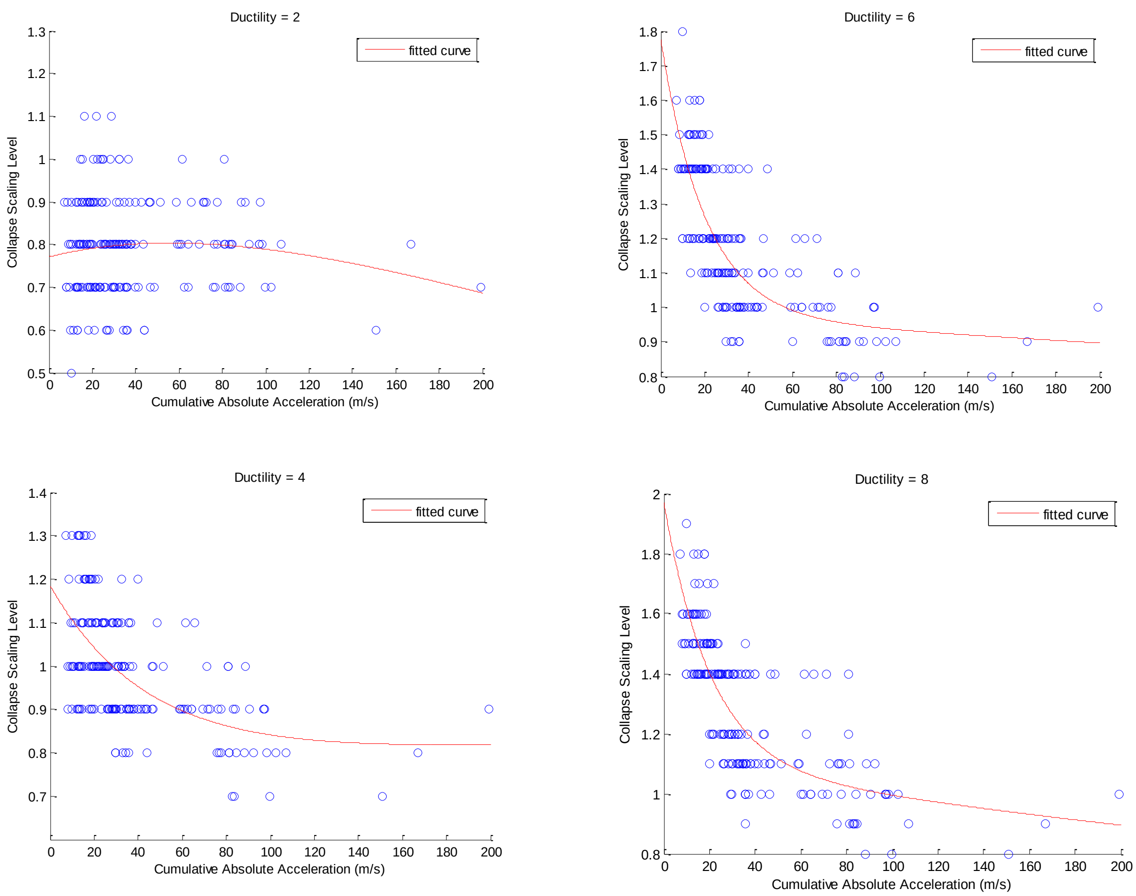


2.4.3 Cumulative Absolute Velocity (CAV)

CAV is defined as seen in Equation 2.4.3. In this equation, a(t) is the acceleration time history recorded during the ground shaking event and T is the total time length of the record. The results are shown in Figure 14.

$$CAV = \int_0^T |a(t)|dt \qquad \text{Equation 2.4.3}$$

Figure 14 – Collapse Scaling Level vs. Cumulative Absolute Velocity as Duration Measure

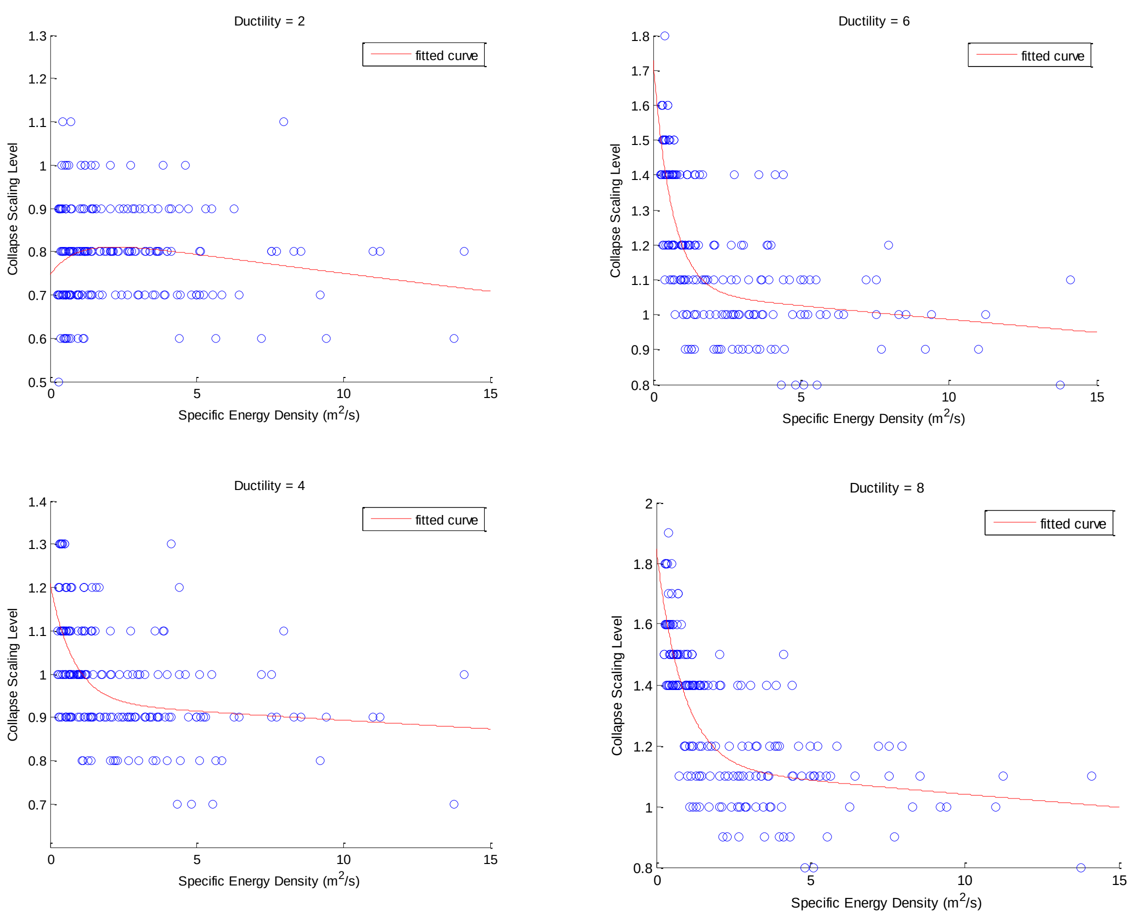


2.4.4 Specific Energy Density (SED)

SED is defined as seen in Equation 2.4.4. In this equation, v(t) is the velocity time history recorded during the ground shaking event and T is the total time length of the record. The results are shown in Figure 15. One disadvantage of the SED is m^2/s output unit because it is not a familiar physical quantity.

$$SED = \int_0^T v(t)^2 dt \qquad \text{Equation 2.4.4}$$

Figure 15 – Collapse Scaling Level vs. Specific Energy Density as Duration Measure

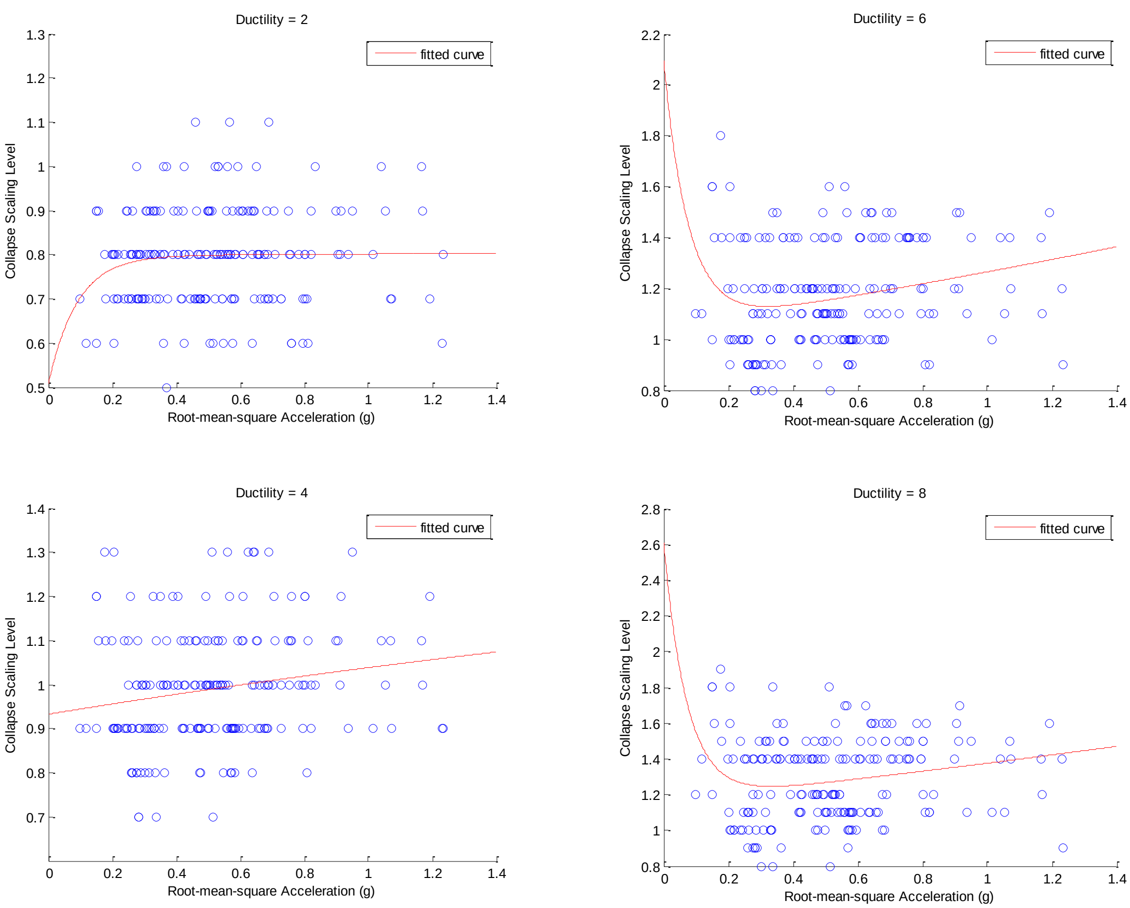


2.4.5 Root Mean Square Acceleration

Root mean square acceleration is defined as seen in Equation 2.4.5. In this equation, a(t) is the acceleration time history recorded during the ground shaking event and T is the total time length of the record. The results are shown in Figure 16. There is no trend observed between the collapse scaling level and SED as a duration measure.

$$a_{RMS} = \sqrt{\frac{1}{T}\int_0^T a(t)^2 dt} \qquad \text{Equation 2.4.5}$$

Figure 16 – Collapse Scaling Level vs. Specific Energy Density as Duration Measure

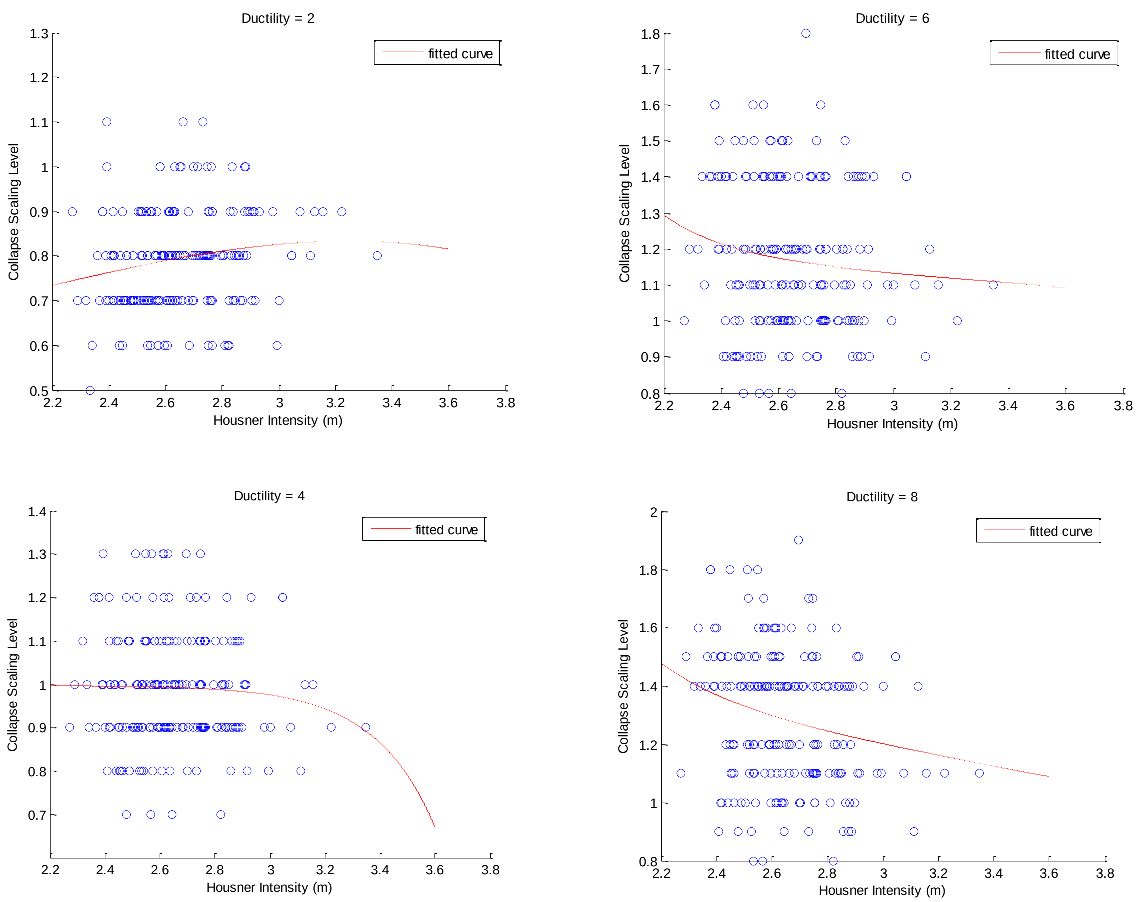


2.4.6 Housner Intensity

Housner Intensity is defined as seen in Equation 2.4.6. In this equation, the spectral velocity of the ground motion, S_v, is integrate over a range of periods, usually from 0 second to 10 seconds. A 5% viscous damping, ξ, is assumed. The results are shown in Figure 17. It is rather inconvenient to consider a length unit, metre, as a duration measure.

$$SI(\zeta) = \int_0^{10} S_v(T,\zeta)\,dT \qquad \zeta = 5\% \qquad \text{Equation 2.4.6}$$

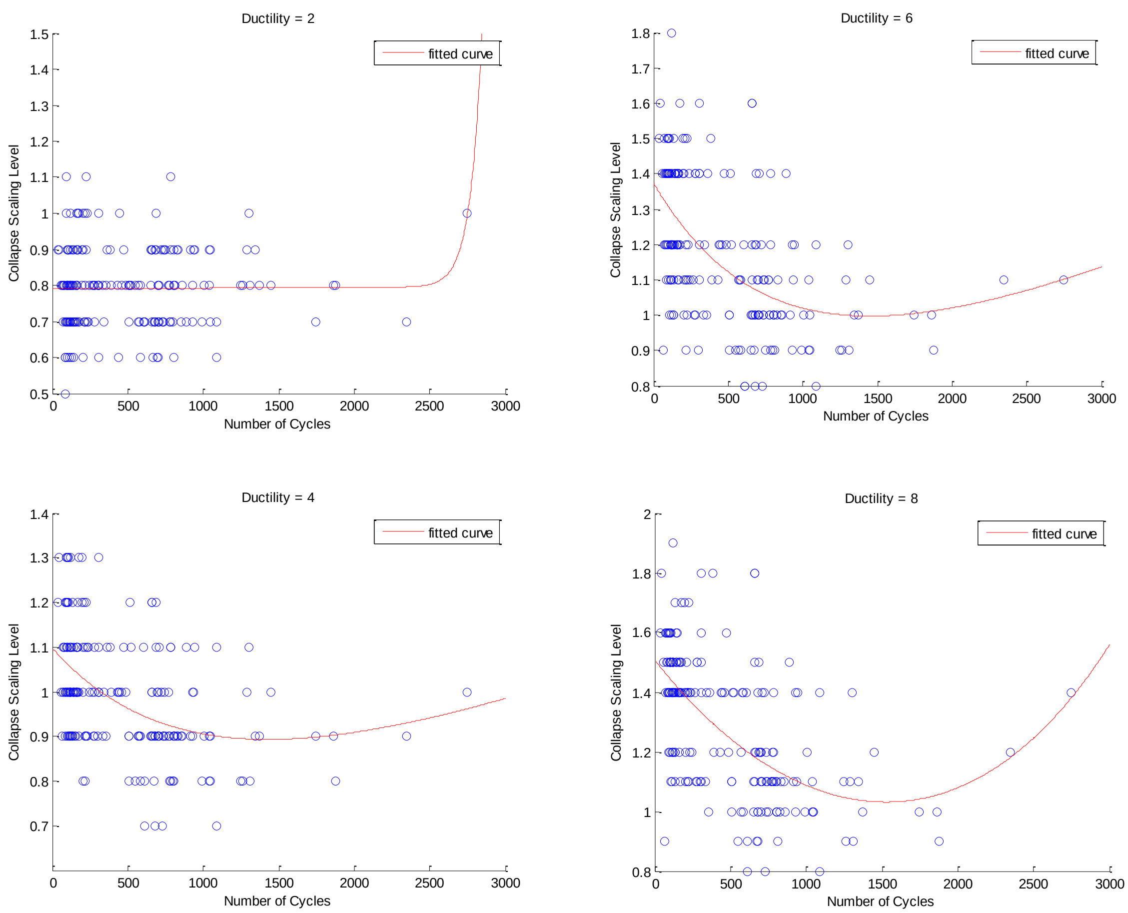
Figure 17 – Collapse Scaling Level vs. Specific Energy Density as Duration Measure



2.4.7 Number of Cycles

The number of cycles is simply a count of how many times the acceleration time history record being in motion back and forth. The results are shown in Figure 18. There is also no trend observed between the collapse scaling level and the number cycles as a duration measure.

Figure 18 – Collapse Scaling Level vs. Number of Cycles as Duration Measure



2.5 Advantage of Significant Duration

The advantage of using signification duration to represent ground motion duration is two-fold. First, as demonstrated in Section 2.4, significant duration does show a trend between the collapse scaling level and ground motion duration. When the ductility of the system is low, the collapse scaling level is unaffected by duration. As the ductility of the system increases, the collapse scaling level decreases. This trend is comparable to the results generated using CAV or SED as a duration measure. Second, significant duration is vastly adopted in the research community. To make the study results compatible with those presented in the Literature Review, signification duration is selected as the duration metric.

Chapter 3: Research Methodology

All study results are generated by computer simulation. The academic software Open System for Earthquake Engineering Simulation version 2.4.6 (OpenSees, 2014) is used because of its high capability in performing nonlinear structural analysis.

3.1 Overview of Research Methodology

Structural collapse assessment using incremental dynamic analysis is the main tool for the research. Models of a reinforced concrete shear wall and a concrete frame building are developed separately. These models are then subjected to two sets of short and long duration ground motions. For each ground motion record, a constant multiplier is applied to simulate 10% to 250% of the record intensity. At each run, the intensity at which collapse is first detected is recorded. Collapse is defined as the drift level where the incremental dynamic analysis curves become flat and small changes in ground motion scaling causes extreme changes in drift. Then for each set of ground motion, the average and standard deviation of the intensity are computed. By assuming a lognormal distribution, the cumulative probability of drift exceedance at increasing intensity level can be established. Such plot is termed fragility curve in this thesis.

3.2 Techniques in Structural Dynamic Analysis

There are four types of structural analysis procedures in general. These procedures are further broken down into two categories: static or dynamic, then linear or nonlinear within each category. Table 1 summarizes the four types of analysis and their uses.

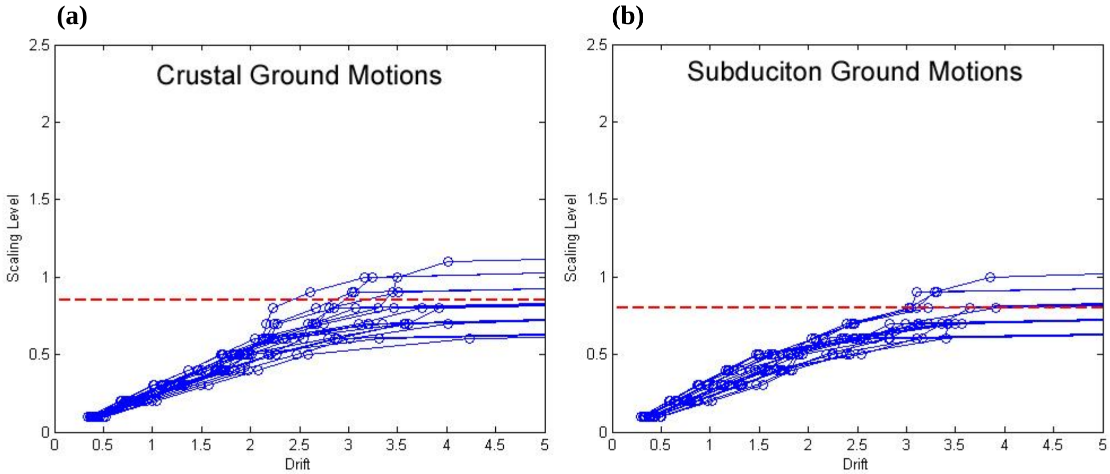
Table 1 – Matrix of Four Types of Structural Analysis

	Linear	**Nonlinear**
Static	Example: Elastic Beam Theory Use: typical tool to solve for structural internal forces and reactions given externally applied loads	Example: Pushover Analysis Use: plot of deformation versus applied force to examine the ductility of a structure
Dynamic	Example: Modal Analysis Use: a quick assessment of the dynamic forces within a structure, also provides as estimation of a structure's modal periods	Example: Time History Analysis Use: the state-of-the-art simulation of the response of a structure under an applied ground motion, fundamental to the Incremental Dynamic Analysis

3.3 Incremental Dynamic Analysis (IDA)

IDA is a series of time history analysis with a slight increase in the ground motion intensity at each run (Vamvatsiko and Cornell, 2002). During each run, key engineering parameters such as interstorey drifts or base shears are recorded. The ground motion record is then scaled up and the time history analysis is repeated until dynamic instability in the analysis software occurs, usually in the form of large interstorey drift at small scaling increment. An example of the IDA results is presented in Figure 19.

Figure 19 – Sample IDA Results (a) Crustal Ground Motions (b) Subduction Ground Motions



3.3.1 Intensity Measure

In this study, the intensity measure is chosen as a constant multiplier ranging from 0.1 to 2.5 at 0.1 increments to simulate 10% to 250% at 10% increment of the ground motion record after spectral matching.

3.3.2 Damage Measure

Interstorey drift ratio is the damage measure used in all studies. This damage measure is chosen over the other because the study results shall be comparable to the Seismic Retrofit Guidelines 2nd Edition (SRG II, 2013). SRG II is part of the British Columbia Ministry of Education program to reduce overall seismic risk of public schools. In light of performance-based design methodology, the philosophy is to prevent loss of lives by reducing the probability of collapse, rather than preventing damages from occurring. Interstorey drift ratio is chosen to be the indicator of building performance in SRG II. Following the state-of-the-art procedure for simulating dynamic building responses, multiple nonlinear time history analyses are conducted on each building system to establish the probability of exceeding the deformation limit state. The purpose of nonlinear analysis in SRG II is to determine an appropriate level of lateral strength, expressed as some portion of the structural weight, such that the probability of drift exceedance at the life safety performance criterion is kept low at 2% in 50 years.

3.4 Fragility Curves from IDA

The scaling level at which collapse occurs is treated as a random variable. The assumption is that this random variable follows the lognormal distribution, so that the cumulative probability of collapse, which in this case is drift exceedance, can be plotted. The scaling level at which drift exceedance occurs is recorded. This natural logarithm value of this scaling level is then computed. The mean and standard deviation of these natural logarithmic scaling levels then become the input arguments in the lognormal distribution plot. One of such plot, referred to as a fragility curve in this thesis, is shown in Figure 20.

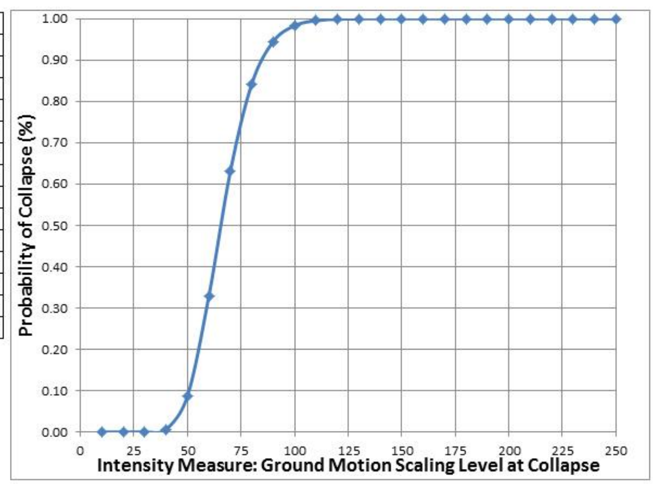
Figure 20 – Creation of Fragility Curve from IDA Results.

IDA Results

EQ Index	Scaling Intensity at Collapse (SI)	ln(SI)
1	50	3.912
2	90	4.500
3	80	4.382
4	70	4.248
5	60	4.094
6	70	4.248
7	70	4.248
8	60	4.094
9	60	4.094
10	70	4.248
11	60	4.094
12	70	4.248
13	40	3.689
14	80	4.382
15	70	4.248
	Mean =	4.182
	Standard Deviation =	0.199

Fragility Curve

X-Axis	Y-Axis	X-Axis	Y-Axis
10	1.76E-21	130	1.00E+00
20	1.24E-09	140	1.00E+00
30	4.34E-05	150	1.00E+00
40	6.58E-03	160	1.00E+00
50	8.73E-02	170	1.00E+00
60	3.29E-01	180	1.00E+00
70	6.30E-01	190	1.00E+00
80	8.42E-01	200	1.00E+00
90	9.45E-01	210	1.00E+00
100	9.83E-01	220	1.00E+00
110	9.95E-01	230	1.00E+00
120	9.99E-01	240	1.00E+00
		250	1.00E+00



In Figure 20, the IDA Results Table represents the results obtained from an IDA. The first column, EQ Index, is a generic number for ground motion tracking purposes. The second column, Scaling Intensity at Collapse (SI), represents the constant multiplier as described in Section 3.3.1, multiplied by 100 to allow for the natural logarithm operation in the third column. The mean and standard deviation are calculated statically using the natural logarithmic values, assuming these values follow the normal distribution. The Fragility

Curve Table lists the x- and y- axis values for plotting the fragility curve. The x-axis represents the Intensity Measure corresponding to the constant multiplier described in Section 3.3.1, multiplied by 100 to account for how the mean and standard deviation are calculated. The y-axis represents the lognormal distribution of the Intensity Measure, using the calculated mean and standard deviation from the IDA Results. Fragility curves generated in this thesis are made with a spreadsheet program with built-in statistical functions.

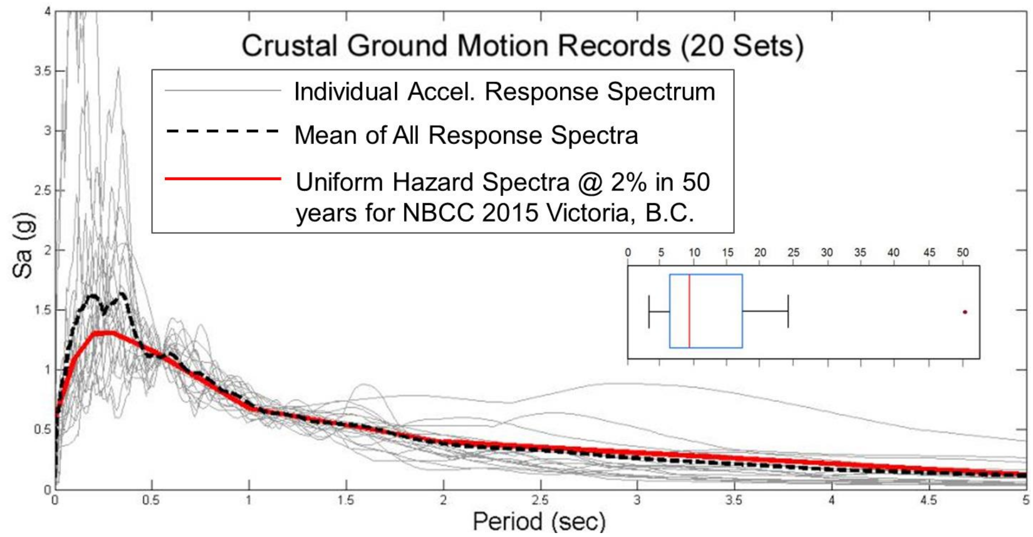
3.5 Ground Motion Selection and Scaling

Ground motion records are selected and scaled to the NBCC 2015 uniform hazard spectrum (UHS) of Victoria, B.C. shown previously in Figure 8. Using the S2GM database (Bebamzadeh et al., 2015), two suites of twenty ground motions are selected for crustal and subduction earthquakes respectively. The selection algorithm in the S2GM database automatically scales the ground motion records to match with the Victoria UHS between periods of 0.2 seconds to 1.5 seconds. This process is known as spectral matching. Then the algorithm will select the top twenty ground motion records that give the best match between the mean response spectrum and the UHS.

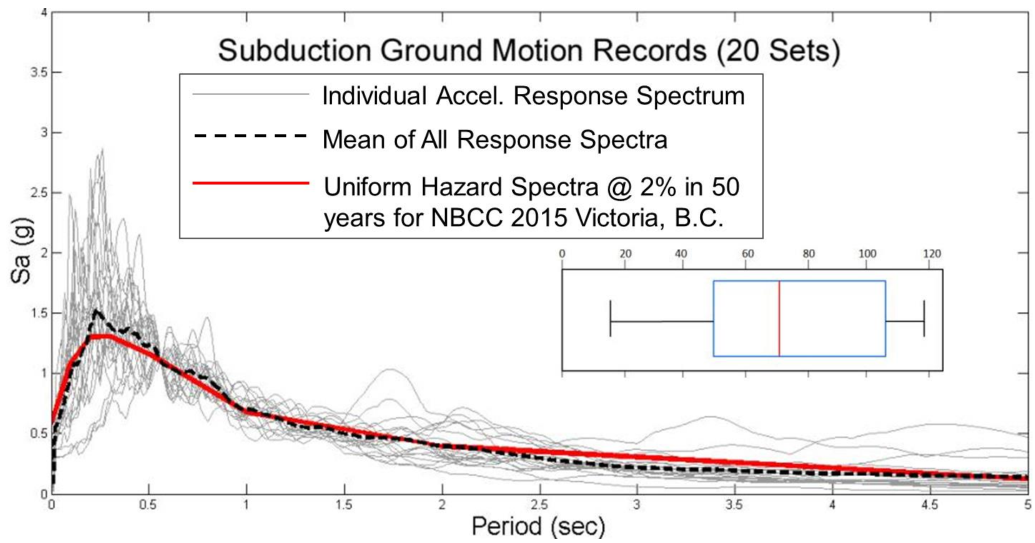
With reference to Figure 21, the NBCC 2015 Victoria UHS is highlighted in red, and the black dashed line represents the mean response spectrum of the selected records, which are shown in grey. The bar-graph insert summarizes the distribution of their significant durations. The crustal suite has an average significant duration of 10 seconds, and 70 seconds for the subduction suite. The terms, crustal ground motion and short duration ground motion, are used interchangeably in this thesis; and the terms, subduction ground motion and long duration ground motion, are also used interchangeable.

Figure 21 – Ground Motions used in this Thesis. (a) Crustal (b) Subduction

(a)



(b)



Chapter 4: Study of a Reinforced Concrete Shear Wall

4.1 Purpose of the Study

By examining a hypothetical concrete shear wall structure, the following items can be established:

(1) Nonlinear modeling capability of OpenSees

(2) Effects of material deterioration on IDA results between short and long duration ground motions

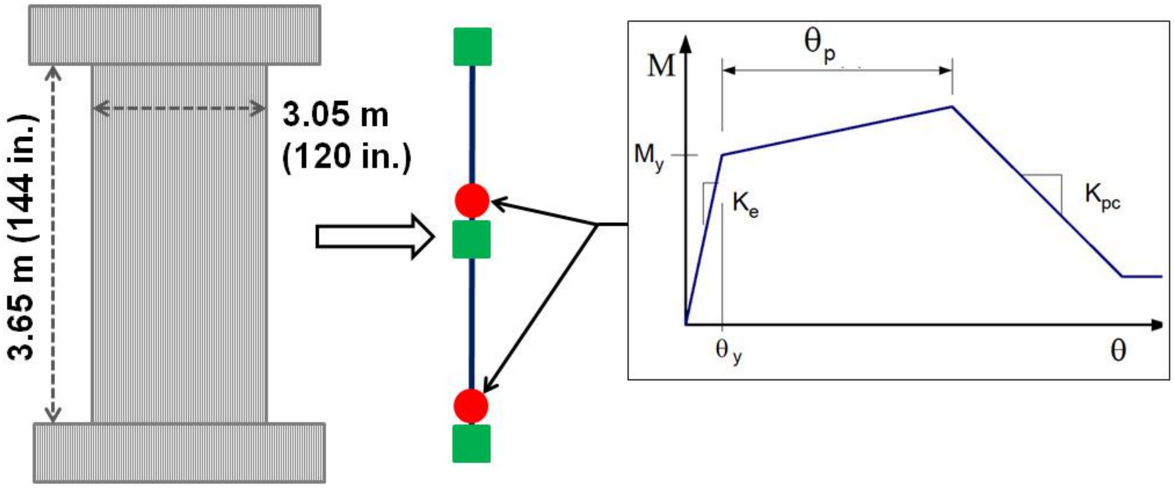
(3) Effects of system ductility on IDA results between short and long duration ground motions

(4) General trends of duration effects on fragility curve

4.2 Model Description

A reinforced concrete shear wall is idealized as a lumped plasticity two-degree-of-freedom (2-DOF) model as shown in Figure 22. Dimension and loading condition of the shear wall are determined in light of Birely (2012) analysis on specimen PW1, whereas the backbone curve for the plastic hinges and hysteresis parameters are selected based on the SRG II Prototype C-6 Shear Wall. The fundamental period of the wall is found to be 0.18 seconds by modal analysis.

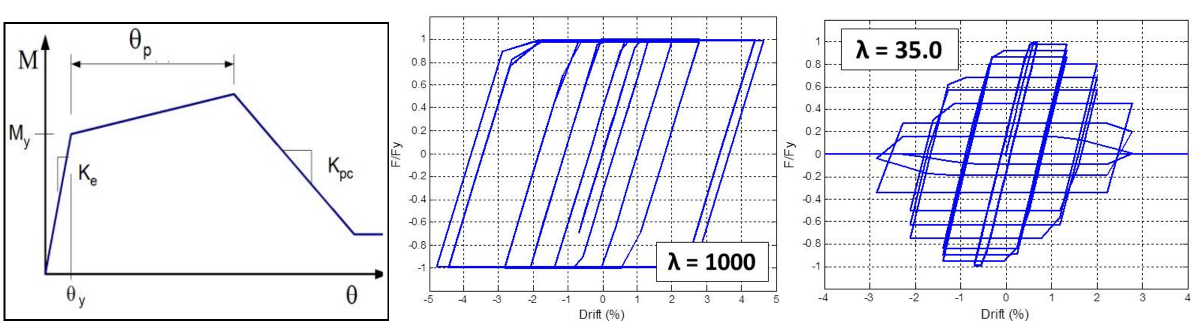
Figure 22 – Idealization Model of the Reinforced Concrete Shear Wall



4.3 Plastic Hinge Backbone Model

The capability of a hysteretic model to capture different forms of deterioration is crucial in seismic collapse assessment (Ibarra, et.al., 2005). Through careful calibration against experimental results, Ibarra (2005) has developed three material models that address both monotonic and cyclic deterioration. These models are subsequently modified by Lignos and Krawinkler (2012) and become the OpenSees uniaxial material model “Bilin”, “ModIMKPeakOriented”, and “ModIMKPinching”. Figure 23 illustrates schematically the deterioration capability of the OpenSees “Bilin” model used in this study.

Figure 23 – Schematic Representation of the Material Hysteretic Degradation Behaviour



4.3.1 Degradation Parameter

Monotonic deterioration refers to strength loss in one cycle. The negative post-capping stiffness on the element backbone curve best exemplifies such deterioration. This negative stiffness is critical for seismic collapse simulation (Ibarra et. al., 2005). Pushover analysis is a common tool to quantify the monotonic deterioration of a structural component.

Cyclic deterioration refers to strength loss in subsequent cycles of loading while the tangent stiffness remains positive. Physically, this behaviour is the result of the element disintegrating under multiple reversing cycles such as concrete spalling or fasteners pulling out. This thesis follows the material deterioration assessment and nonlinear modeling recommendations found in the PEER/ATC-72-1 Report. Loading protocol testing is an effective way to experimentally quantify the amount of cyclic deterioration on structural components. In the OpenSees "Bilin" material model, the amount of deterioration is controlled by four λ parameters that dictate the percentage drop of strength and stiffness using a series of empirical relationships (Lignos and Krawinkler, 2012). The degradation controlling parameter, λ, is set to be 35.0 for all four modes of degradation. This setting is consistent with a former study by Liel and Raghunandan (2013) on concrete frame structure.

4.3.2 Pushover Ductility

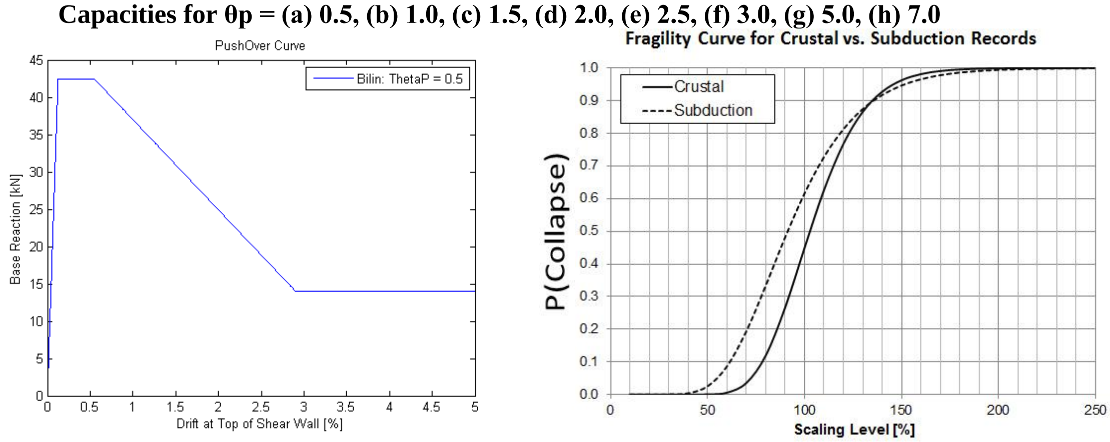
The plastic deformation capacity of the lumped hinge, θ_P, directly contributes to the overall structural ductility. To study the effect of ductility on ground motion duration, θ_P is varied to create hypothetical structural system with ductility ranging from 2 to 9. The fragility curve is generated at each ductility level for each set of the crustal and subduction ground motions.

4.4 Fragility Curve Results

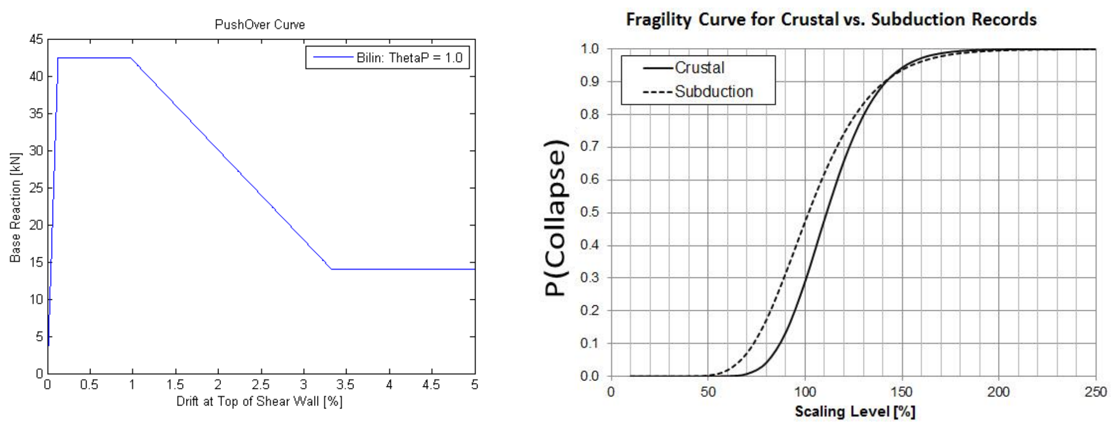
Following the Methodology described in Section 3.0, results from pushover analysis and fragility curves for the idealized reinforced concrete wall are shown in Figure 24.

Figure 24 – Comparison of Pushover and Fragility Curves at Various Plastic Hinge Deformation Capacities for θp = (a) 0.5, (b) 1.0, (c) 1.5, (d) 2.0, (e) 2.5, (f) 3.0, (g) 5.0, (h) 7.0

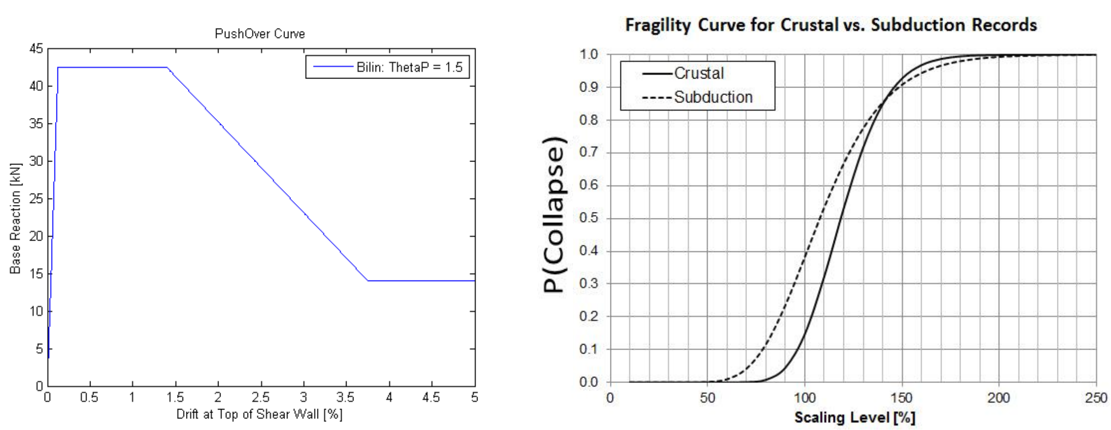
(a)



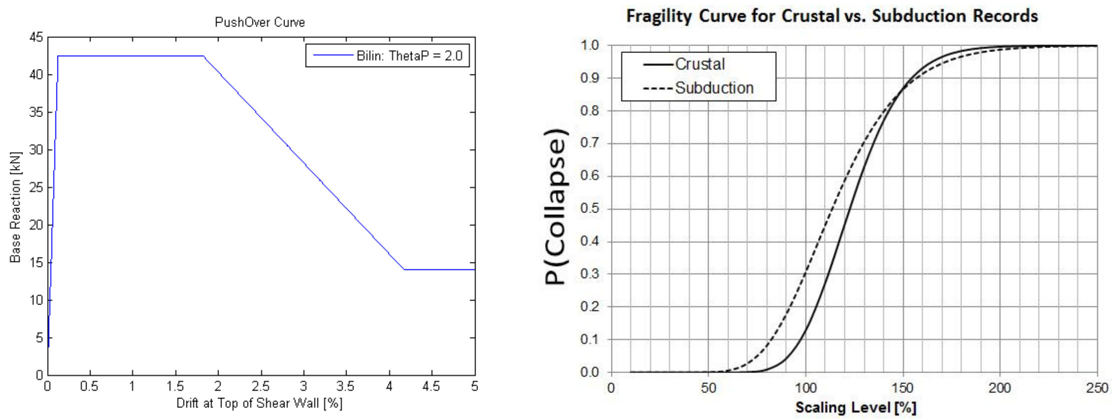
(b)



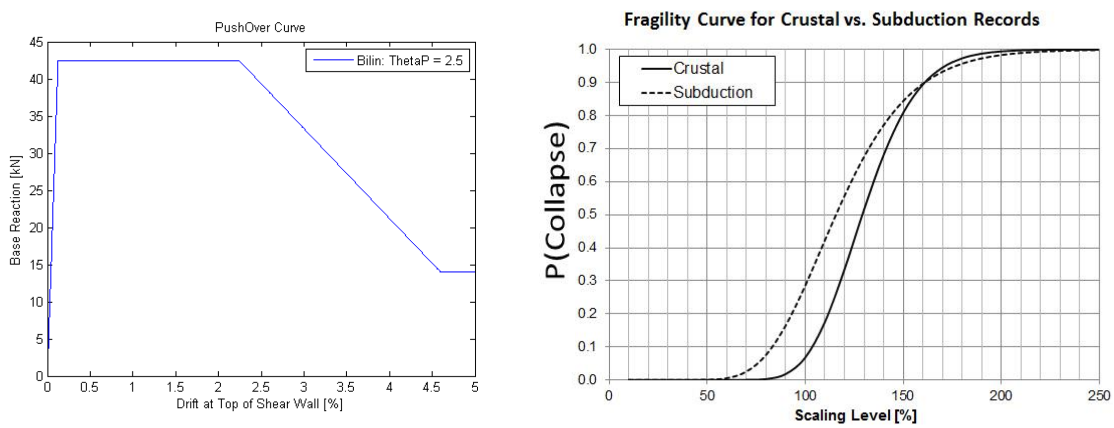
(c)



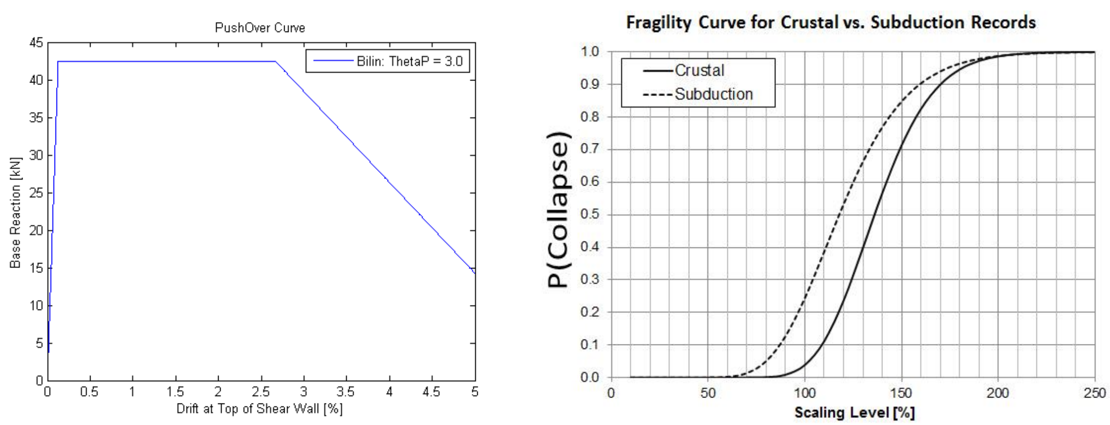
(d)


PushOver Curve
Bilin: ThetaP = 2.0
Base Reaction [kN]
Drift at Top of Shear Wall [%]
Fragility Curve for Crustal vs. Subduction Records
Crustal
Subduction
P(Collapse)
Scaling Level [%]

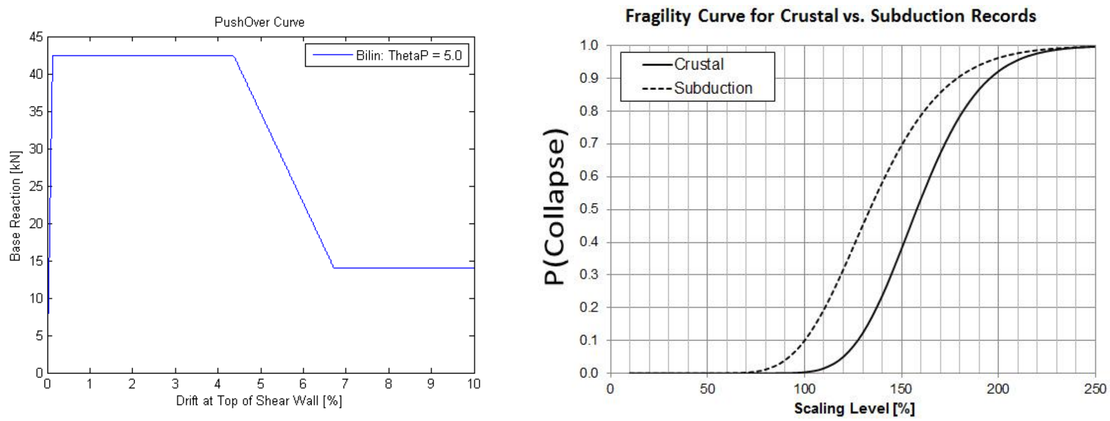
(e)


PushOver Curve
Bilin: ThetaP = 2.5
Base Reaction [kN]
Drift at Top of Shear Wall [%]
Fragility Curve for Crustal vs. Subduction Records
Crustal
Subduction
P(Collapse)
Scaling Level [%]

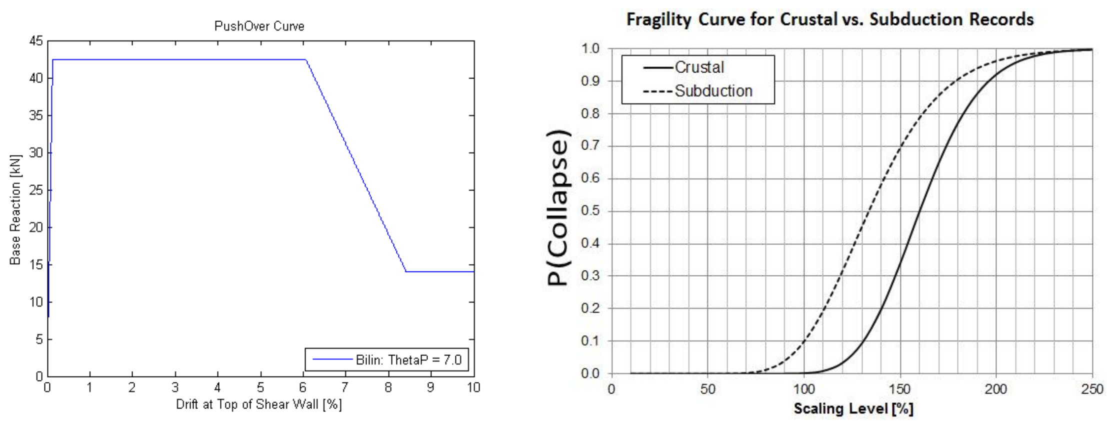
(f)


PushOver Curve
Bilin: ThetaP = 3.0
Base Reaction [kN]
Drift at Top of Shear Wall [%]
Fragility Curve for Crustal vs. Subduction Records
Crustal
Subduction
P(Collapse)
Scaling Level [%]

(g)



(h)



4.5 Results Discussion

Each point on the fragility curve represents the probability of collapse when the ground motion intensity reaches a certain level. There are two key trends observed from these results: the decreasing probability of collapse as the plastic hinge deformation capacity increases, and the fundamental difference in probability of collapse between crustal and subduction ground motions at the same intensity level.

As the plastic hinge deformation capacity, θ_P, increases, so does the overall ductility of the structure. The enhanced capability in enduring larger deformation allows the structure to reach larger interstorey drifts before collapse while the plastic zone dissipates the input earthquake energy. Such enhancement is most evident at lower ductility levels where θ_P ranges from 0.5 to 3.0. Under the same hazard, or when the scaling level is 100%, the probability of collapse drops from 0.45, when θ_P equals 0.5, to 0.05, when θ_P equals 3.0. Therefore, enhancing the ductility of a structure may be one effective way to lower the probability of collapse. The trend of decreasing probability of collapse is less evident at larger ductility levels where θ_P ranges from 5.0 to 7.0, because the huge deformation capacity cannot be utilized before the model loses too much of its strength capacity.

Another interesting observation is the difference between crustal and subduction records. At each plastic hinge deformation capacity level, it takes a less intense subduction ground motion to cause the same probability of collapse. One possible explanation is the higher number of cycles in the subduction records that may have exhausted the pre-defined deterioration capability of the material model, represented by the parameter λ. Deformation capacity and rate of deterioration are therefore two key parameters that one must carefully calibrate to produce acceptable results.

Chapter 5: Study of Reinforced Concrete Frame Building

5.1 Purposes of the Study

A fictitious reinforced concrete frame building is designed in light of modern design code (ATC 78-1, 2012). This building is then made into a computer model for academic research. By adjusting the model hysteretic parameters and repeating the IDA procedure as described in Section 3.0, the following items are investigated:

(1) Structural responses of a complex building under crustal and subduction ground motion versus the simple wall in Section 4.0

(2) Effects of material deterioration on IDA results between short and long duration ground motions for a complex building

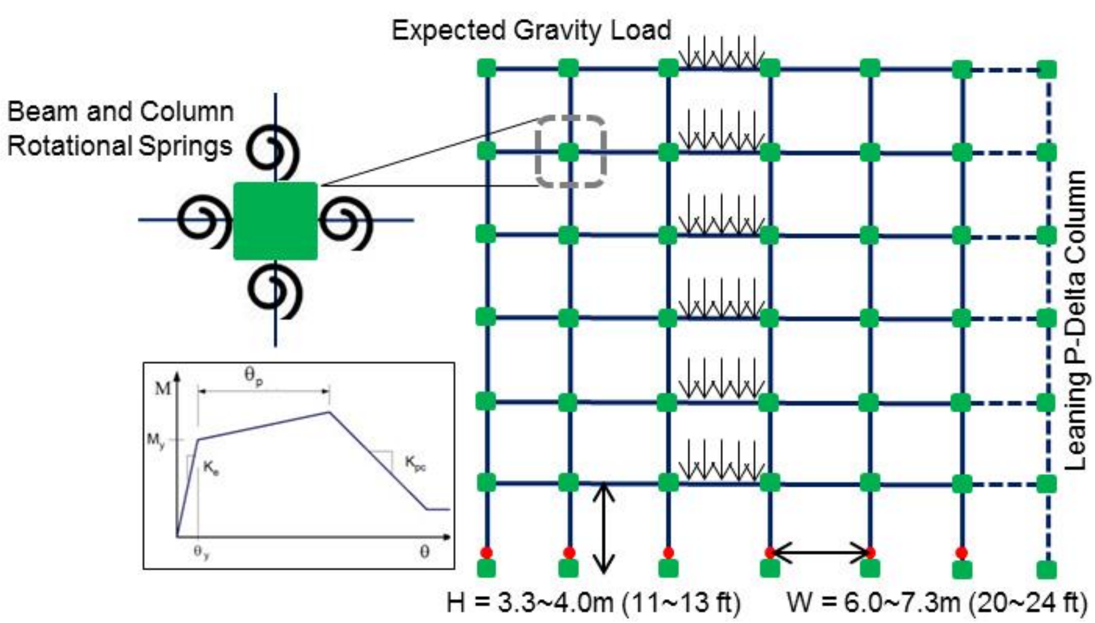
(3) Effects of system ductility on IDA results between short and long duration ground motions for a complex building

5.2 Model Description

This study makes use of a ductile concrete moment frame model developed by the Applied Technology Council (ATC 78-1, 2012). The five-bay-and-six-storey building was designed to satisfy ductile reinforcement detailing requirements in modern building codes that are very much alike between the United States and Canada. The design base shear is approximately 10% of the effective seismic weight of the building, and the fictitious building is designed for a site located at Seattle, Washington, USA. The building is 20.72 m tall. Using the empirical period prediction equation for concrete building in the NBCC, $0.075(h_n)^{3/4}$, and the seismic design spectrum for a Site Class C location in Victoria, B.C., Canada, the design base shear is estimated to be between 9.1% and 13.8% of the effective seismic weight of the structure. Therefore, the existing structural member sizes and reinforcement ratios are considered adequate for Victoria, B.C. Figure 25 is a schematic representation of the fictitious reinforced concrete frame. Beams and columns are modeled as lumped plasticity elements with rotational hinges forming at both ends. Since OpenSees was also the analysis program used in the ATC 78-1 (2012) report, the hinges are assigned with element specific monotonic and cyclic parameters as reported by ATC 78-1 (2012). Backbone curve used in the plastic hinges is further described in the subsequent section.

The modeling of the reinforced concrete frame follows well-established guidelines. The ATC 72-1 (2010) document has covered the state-of-the-art methods in modeling frame components. Applicable recommendations in the ATC 72-1 (2010) report have been adopted to construct a "base case" study model using the member sizes and reinforcement ratios as reported in the ATC 78-1 (2012) Prototype 1. The first three structural periods are found to be 1.59s, 0.56s, and 0.29s by modal analysis, and the associating deformed shapes are all global deformations in side-swaying manner.

Figure 25 – Schematic Representation of the Concrete Frame Model



5.2.1 Plastic Hinge Backbone Model

The backbone curves are grouped into two categories: beams and columns. Following the work by Ibarra (2005) as described in Section 4.3, Haselton, et.al. (2008) have performed a parameter calibration study on the OpenSees "Bilin" material model against experimental data, and have summarized their findings in the PEER Report 2007/03 publication. Notably, the team has developed statistic regression equations to predict the two parameters of interest to this thesis – degradation parameter (λ) and deformation capacity (θ_P) – using design information such as structural member sizes and reinforcement ratios. The ATC 78-1 (2012) Prototype 1 in Figure 25 has adopted recommendations by Haselton, et.al. (2008), and the magnitudes of the parameter are as shown in Table 2.

Figure 26 – Backbone Curve Parameter Definition in ATC 78-1 (2012)

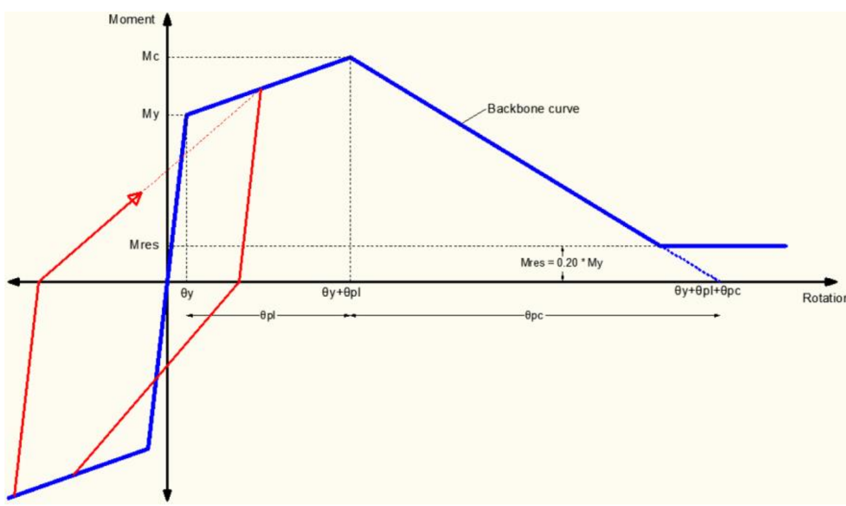


Table 2 – Summary of Backbone Curve Parameters use in "Base Study"

Components	θ_{pl} (rads)	θ_{pc} (rads)	λ
Columns	0.06	0.1	110
Beams	0.05	0.15	125

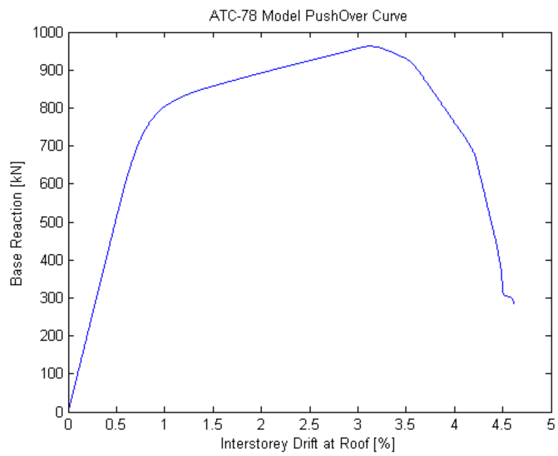
The blue curve in Figure 26 is commonly referred to as the backbone curve. The location at which the kinks in the backbone curve occur can be easily defined in OpenSees "Bilin" material model as input arguments in the Tool Command Language (Tcl) programming script. In Figure 26, θ_y is the rotation in radians at yielding, θ_{pl} is plastic deformation region, and θ_{pc} is the deformation capacity without fracturing. On the vertical axis, M_c is the maximum bending moment, M_y is the moment at yield, and M_{res} is the residual moment capacity without fracturing.

The red curve in Figure 26 traces the path of loading and unloading. While each point along the red curve represents the structural analysis result at one time step of the applied load, the slope between each pair of points is controlled by the degradation parameter, λ. The effect of λ has been shown previously in Figure 23. The study be ATC 78-1 (2012) has used a single value for all four degradation parameters necessary to define the OpenSees "Bilin" material model. This simplification has been adopted in this thesis.

5.2.2 Global Ductility by Pushover Analysis

To confirm the ductile nature of the reinforced concrete frame, static pushover analysis is performed. The plot of base shear reaction versus interstorey drift at roof level is presented in Figure 27. While the pushover curve is rather smooth in the elastic and plastic regions, the post-capping region exhibits extensive capacity loss. The OpenSees solver tends to fail to converge at high non-linearity. The pushover analysis is therefore terminated at the onset of such convergence instability. Similar pushover curves can be obtained if base shear reaction is plotted against interstorey drift at other levels.

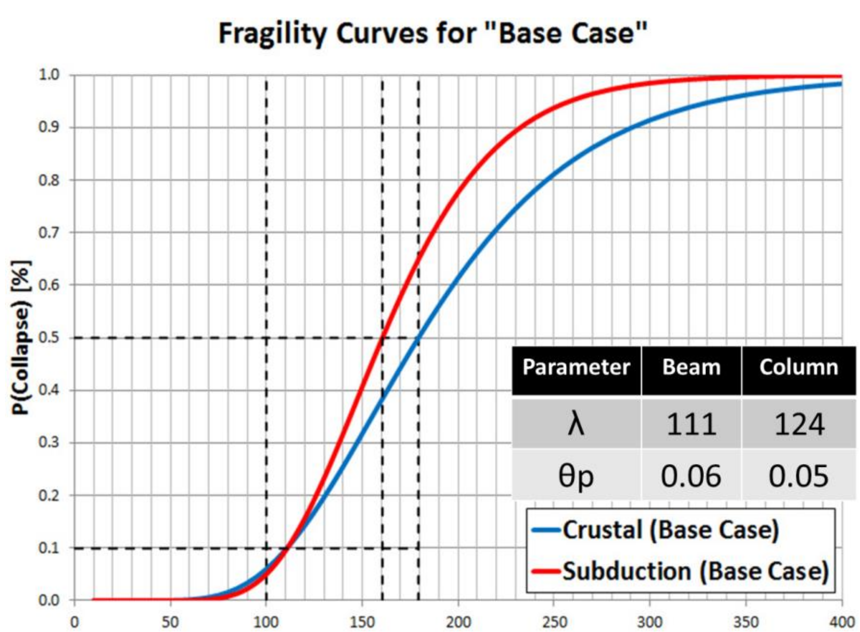
Figure 27 – Reinforced Concrete Frame Pushover Curve for “Base Case” Design



5.3 Base Case Fragility Curve Results

As was the case with the reinforced concrete shear wall in Section 4.0, the “base case” reinforced concrete frame is also subjected the IDA analysis procedure described in Section 3.0. As shown in Figure 28, significant discrepancy between the probability of drift exceedance under crustal and subduction ground motions is also evident in the “base case” frame.

Figure 28 – Fragility Curves for Reinforced Concrete Frame “Base Case”



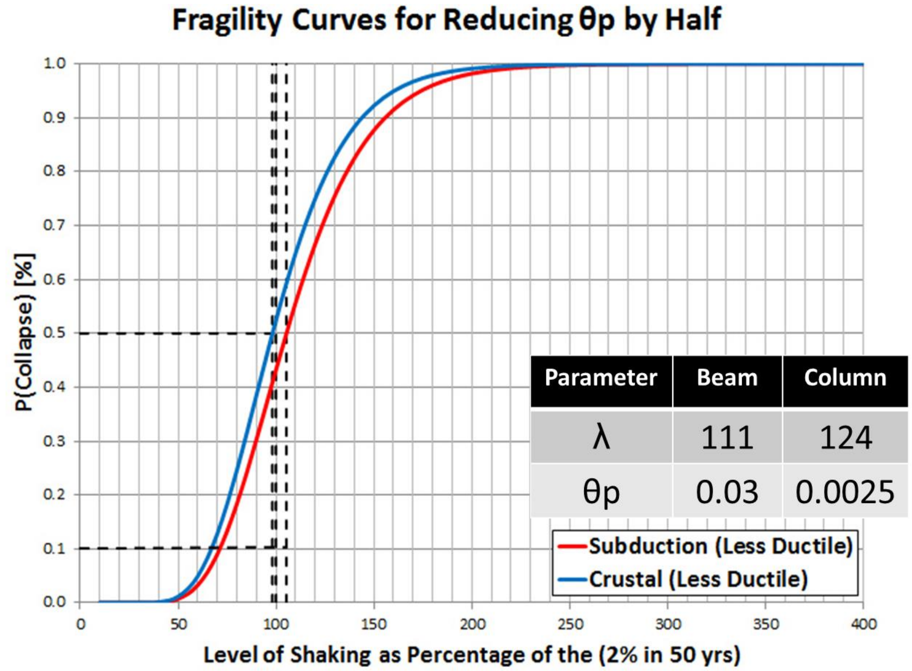
Examination on the plastic hinge hysteresis data reveals that the energy dissipation capacity, or the area under the M-θ curve, depletes more under subduction ground motion. A plausible reason is the higher number of load cycles in a subduction ground motion. More load reversing cycles mean the λ-dependent degradation algorithms are activated more often, thus resulting in lesser area under an M-θ curve at the same rotational deformation.

5.4 Effect of System Ductility on Fragility Curves

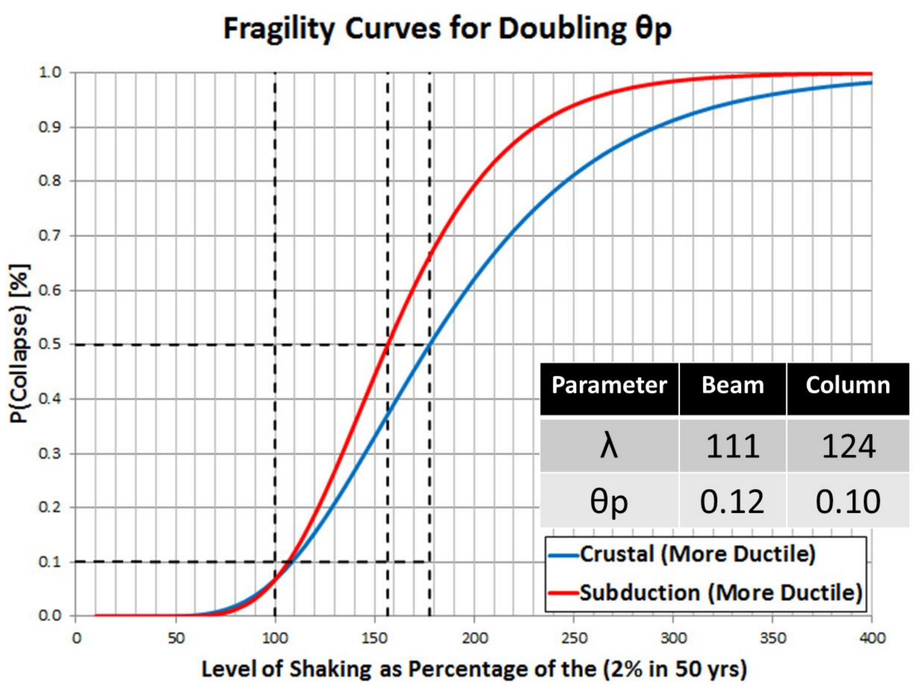
One of the two parameters that control ductility in the model, θ_{pl} in Figure 26, is artificially manipulated to be halved and doubled. IDA is repeated and the fragility curves are as shown in Figure 29 (a) and (b). It should be noted that such change in ductility is completely fictitious and may not be achievable physically. As described in Section 5.2, the current deformation capacity at the plastic hinges is obtained empirically from a statistic regression equation that accounts for member sizes and reinforcement ratios. When θ_{pl} is halved, the member sizes might not necessarily meet the strength requirements, let alone seismic design requirements. When θ_{pl} is doubled, the structure might have been over-designed and is not economical. The post-capping deformation capacity parameter, θ_{pc}, is kept constant as the “base case”. The degradation parameter, λ, is also kept constant.

Figure 29 – Fragility Curve for (a) Less Ductile System (b) More Ductile System

(a)



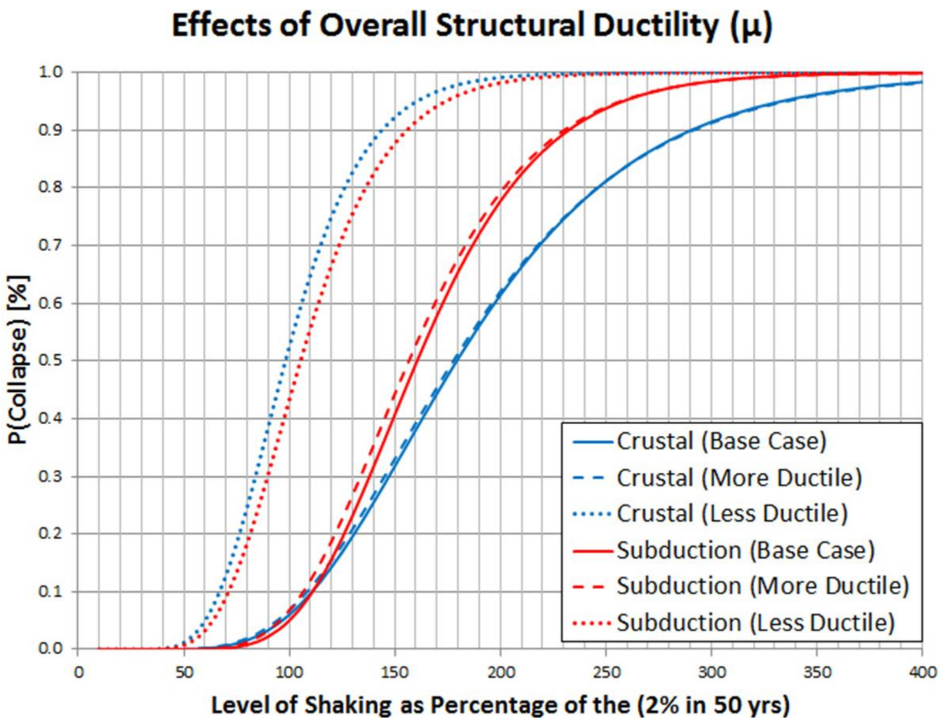
(b)



5.4.1 Comparison of Fragility Curves with Varying Ductility

Fragility curves associating with the two artificially manipulated models are compared to those of the "base case", see Figure 30. A general trend can be observed between the frame fragility curves and the wall fragility curves in Section 4.0; that there is a fundamental difference between the fragility curves generated using crustal and subduction ground motions. Enhancing ductility for a non-ductile system is very effective in shifting the fragility curves towards the right, thus more favourable as the probability of drift exceedance is reduced at any given ground motion intensity. However, enhancing ductility for an already ductile system has barely any effect. This observation is very crucial when considering seismic retrofit work for buildings. The designers shall carry out a ductility assessment to determine if the existing building warrants the addition or replacement of components to enhance the building ductility, and then quantify the benefits with a similar IDA analysis.

Figure 30 – Effects of Overall Structural Ductility on Fragility Curves

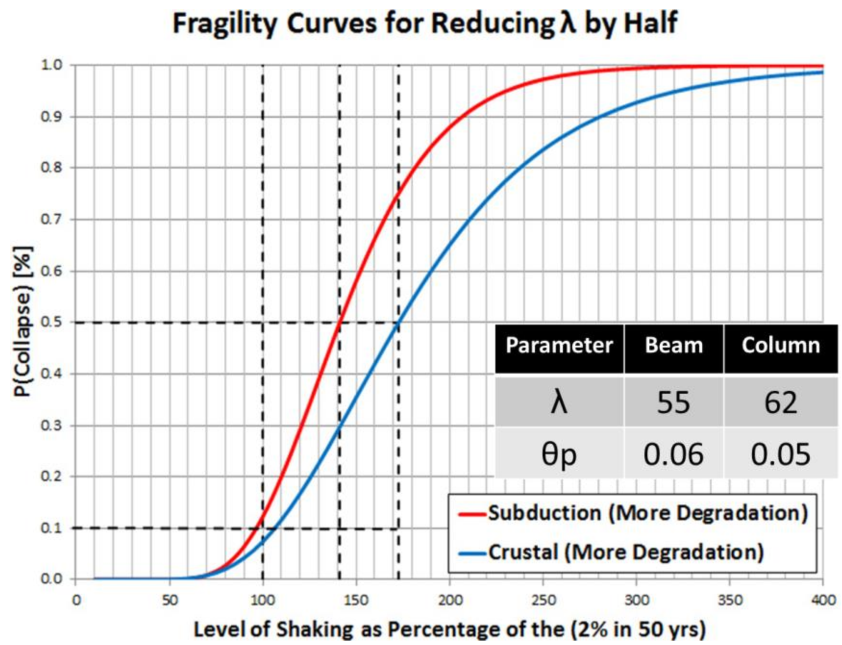


5.5 Effect of Cyclic Degradation on Fragility Curves

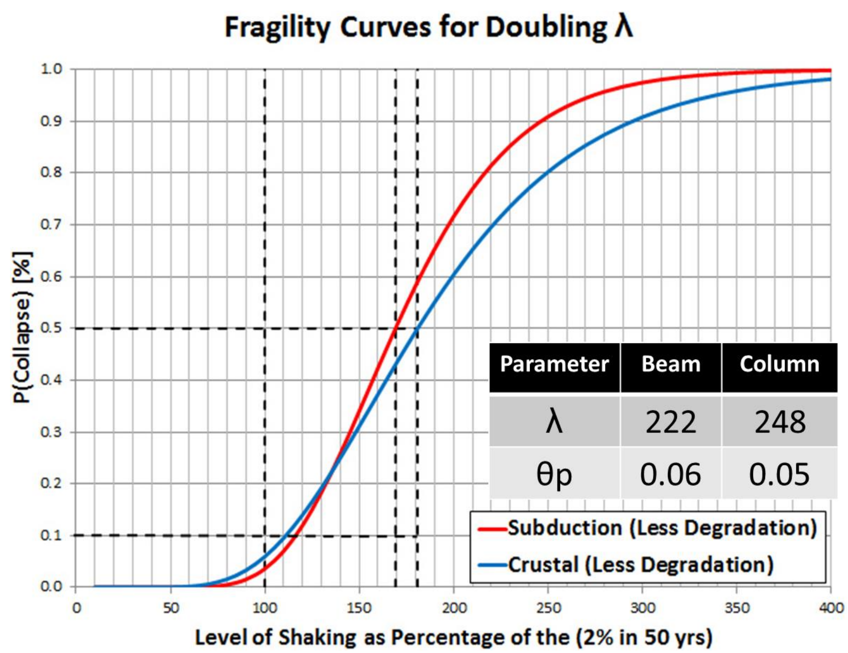
The degradation controlling parameter, λ, is artificially altered to produce lesser and more deterioration in the system, see Figure 31. It should be noted that the amount of degradation is controlled by some mathematical algorithms in OpenSees, and may not necessarily correspond to any physical change in the materials.

Figure 31 – Fragility Curve for (a) Less Deterioration System (b) More Deterioration System

(a)



(b)



5.5.1 Comparison of Fragility Curves with Varying Deterioration

Effects of material degradation on the probability of drift exceedance is not as profound as ductility. Recall from Figure 23 that for the degradation parameter, the higher the λ value, the lesser the degradation. Not surprisingly, a less deteriorating structural system (doubling λ) would have a lower probability of drift exceedance than an equivalent system with more deterioration (halving λ). What is interesting, however, is the influence of material degradation when facing subduction ground motions as opposed to crustal ones. With reference to Figure 32, the amount of degradation does not have much effect on the crustal ground motion suite, whereas the median collapse intensity reduces by almost 10% in the subduction case. The median collapse intensity refers to the ground motion scaling factor, or the constant multiplier described in Section 3.3.1, that results in a probability of drift exceedance of 50% under the lognormal distribution assuption.

At lower probability of drift exceedance, the ground motion intensities between crustal and subduction are reasonably close. At higher probability of drift exceedance, however, the intensities deviate quite dramatically, especially in the case of higher deterioration. The ground motion intensity corresponding to a probability of drift exceedance of 10% and 50% are summarized in Table 3.

Figure 32 – Effects of Material Deterioration on Fragility Curves

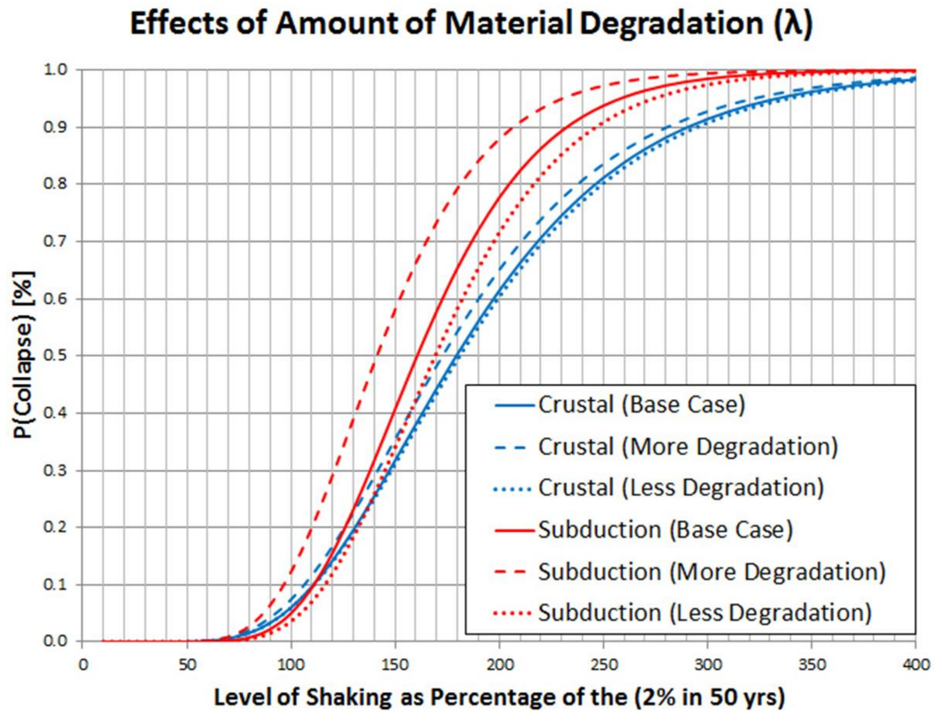


Table 3 – Ground Motion Intensity Corresponding to P(Collapse) = 10% and 50%

P(Collapse) =	10%		50% (Median)	
Case	**Crustal**	**Subduction**	**Crustal**	**Subduction**
"Base Case"	111%	111%	179%	161%
"λ x2"	111%	116%	181%	169%
"λ x 0.5"	106%	97%	173%	141%

Chapter 6: Study Results Discussion

This thesis aims to answer the question what effects do long duration ground motions have on a structure. This chapter provides an in-depth analysis on the topic and identifies reasons to the fragility curve results described in Chapter 4 and Chapter 5. Refinement to the studies and future research needs are also proposed.

6.1 Effects of Long Duration Ground Motion on Probability of Drift Exceedance

A general trend has been established from Chapter 4 and 5. With the incremental dynamic analysis framework in place, a structure is found to be more susceptible to drift exceedance, hence the likelihood of collapse, under the longer duration subduction ground motion. With reference to Figure 24, Figure 30, and Figure 32, when read vertically, the fragility curve pertaining to the longer duration subduction ground motion suite is often above that of the shorter duration. The interpretation is that if a structure is subjected to a long duration ground motion, it is more likely to experience drift exceedance than a short duration ground motion. Putting this interpretation into the geological context of Victoria, B.C., a building is going to suffer more damage from a subduction earthquake than a crustal one, given than both kinds of earthquakes are undoubtedly possible. The reason for this long duration ground motion effect on the vertical reading of the fragility curve may perhaps be attributed to the nature of the ground motions.

6.1.1 Nature of Ground Motions

While there are many attributes to describe a ground motion, the number of load cycles is perhaps the most important to understand the effect of long duration ground motion. Three distinctive “ground motion” types will be examined in ascending order of complexity: (i) sinusoidal, (ii) loading protocol, and (iii) ground motion time history.

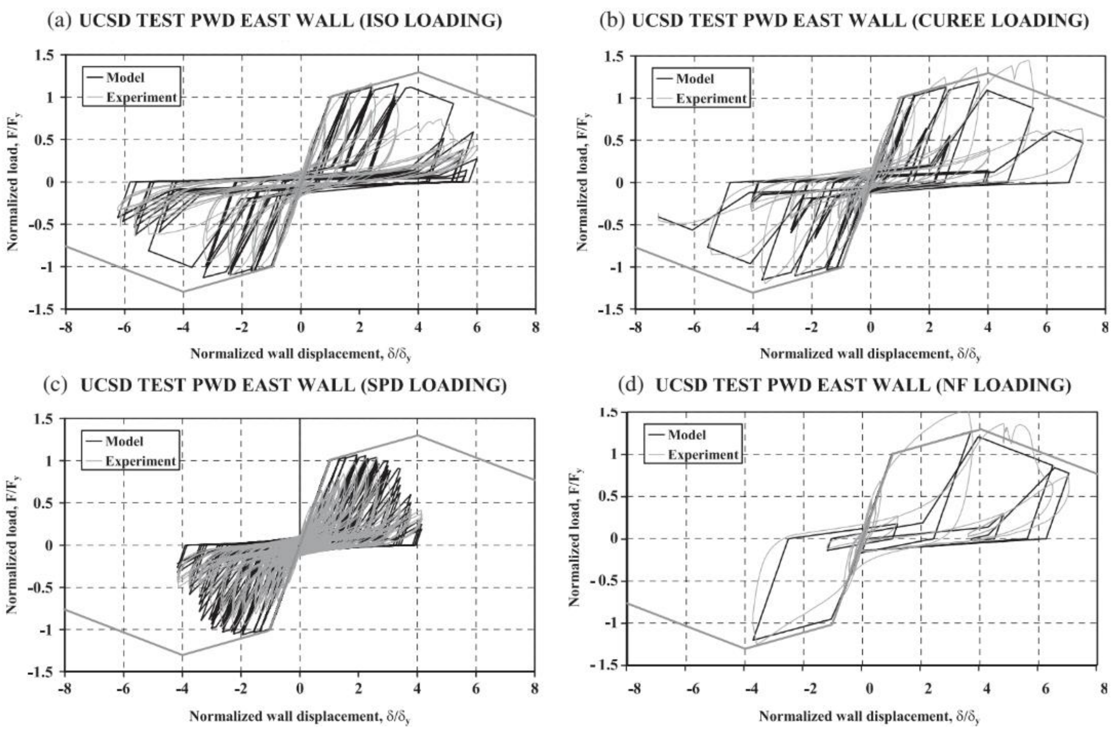
Sinusoidal Motions

All structures can theoretically be failed when subjected indefinitely to a motion strong enough to cause damage. Consider a cantilever reinforced concrete column with some lumped mass at its top where a hydraulic jack will indefinitely push and pull on the mass. This structure is a single degree-of-freedom system under sinusoidal excitation. The bending moment at the base of the column equals to the product of the mass, the acceleration induced by the excitation, and the height of the column. If the column flexural capacity is intentionally made slightly less than the bending demand, the column will start to exhibit damage at its base. Starting from spalling of cover concrete, the damage sequence will also see yielding of longitudinal reinforcement, turning the column behavior non-linear. If the sinusoidal excitation continues indefinitely, perhaps at a few million cycles, the longitudinal reinforcement will fail in fatigue fracture. Assuming the stirrups are still intact, the confined concrete core will then take substantial tensile stress at the extreme fibres, and micro-crack will soon develop and propagate. The column will eventually suffer from a brittle fracture of concrete. In other words, each load cycle brings a structure close to failure. It is therefore intuitive to observe a higher probability of failure in sustained motion.

Loading Protocols

Loading protocols are special excitation sequences that produce visually appealing hysteresis loops. These protocols are typically displacement-based and are used for quantifying the degradation. In the above example, the loss of flexural capacity due to concrete spalling or reinforcement yielding can be quantified using such loading protocols. When Ibarra, et.al. (2005) developed the OpenSees “Bilin” material model, several loading protocols were used, see Figure 33. Degradation is evident in their experimental data under each well-established loading protocol. A structure resistance to motion induced force demands is inarguably diminishing as the number of motion cycles increases. With the degradation parameter, λ, developed by Ibarra, et.al., (2005), the change in the probability of failure in the above column example can be quantified at each load cycle if the hypothetical sinusoidal motion were to sustain.

Figure 33 – Loading Protocols Test Data and Degradation Modeling Comparison (Ibarra, et.al., 2005)



<u>Ground Motion Time History</u>

A ground motion time history is a record of the ground acceleration measured at each time step by an instrument at a certain location. As described in Section 2.4, the signification duration is the time range during which most of the seismic energy is observed within a ground motion time history. Using the above column example, the hysteresis loop will not look as nice as those produced by a loading protocol, and the amount of degradation will not be as easily predictable as in the sinusoidal motion.

Recall in Section 3.5, the spectral values within a certain period range were the basis of ground motion selection and scaling used in this thesis; whereas in Section 2.4.7, the number of cycles and collapse scaling level were found to be not co-related based on a large sample pool. It should be noted that the number of cycles and the maximum spectral values too, may not co-relate. This argument is particularly true for several ground motion time histories pertaining to the 2011 Tohoku Earthquake in Japan. These ground motions are part of the subduction suite used in this thesis, as shown before in Figure 11. Although these Tohoku ground motions fit the selection and scaling procedure, the spectral “bumps” at long period in Figure 11 seems to indicate another influential frequency content. One explanation is the ground motion time history did not record a single seismic event, but rather a series of rupture at different locations that happened to coincide at the site where the measurements were taken. The Tohoku ground motions are in general much longer than other crustal earthquake records from California, USA, or Taiwan, even though all of which comply with the ground motion selection procedure. Effects of these spectral “bumps” on the structure may have been crucial after the structure model becomes non-linear. With the material degradation parameter in place, the period of the structural model is constantly changing at each time step. This phenomenon is known as period elongation. Structure with longer periods are more prone to excitation of longer period. These Tohoku Earthquake spectral “bumps” might have unexpectedly induced additional demands in comparison to the ground motion crustal counterpart, resulting in the further difference in the fragility curve results.

6.2 Effect of Long Duration Ground Motion on Shape of Fragility Curves

Fragility curves are typically parallel for two sets of truly spectral-matched ground motions. While the stretch of the fragility curve is largely determined by the mean and standard deviation used in the lognormal distribution assumption, these means and standard deviations are the output of an IDA operation. Consider the twenty short duration crustal ground motion records used in this thesis. If each record repeats itself several times as the spectrally matching long duration counterpart, the fragility curves for the crustal and the new repeated crustal suite shall theoretically be parallel of one another. An analogous observation can be made in Figure 29 (a) where the crustal and subduction ground motion fragility curves are very close to parallel, merely because the structural model fails very early in the lack of ductility. In contrast, the fragility curves presented in Figure 24, Figure 30, and Figure 32 are rather non-parallel. The effect of long duration ground motion on the shape of fragility curves may perhaps be explained by the mathematical background when performing the IDA.

6.2.1 Mathematical Omissions to IDA Methodology

Structural dynamic analysis is an elegant quest for a solution to the differential equation $m\ddot{x}(t)+c\dot{x}(t)+kx(t)= F(t)$ that idealizes a dynamic equilibrium. For the above SDOF column example, when the excitation input F(t) is a sinusoidal wave, the mathematically efficient solution to the now second degree ordinary differential equation is an exponential function. When F(t) is a ground motion time history applied to an MDOF structure, rigorous solution algorithms are available in OpenSees to efficient approximate the solution to what is now a matrix of differential equations. In the IDA methodology used in this thesis, one matrix of differential equations is solved for each ground motion record at each scaling level. The solver may not necessarily converge to a solution especially after reaching the post yielding capping point in the hysteretic model. This observation is crucial to the long duration ground motion suite because its higher number of cycles in each record means the non-convergence problem will occur easier in an already degraded model. The result is for the longer duration subduction ground motion suite to reach the IDA curve plateau sooner than the crustal set as shown previously in Figure 19.

Two criteria were set to terminate the solution algorithm. The first criterion applies when the resulting interstorey drift reaches a pre-defined threshold of 0.5%. This threshold is higher than the usual vertical deflection limit of H/500, or 0.2%, used in building design because a larger interstorey drift limit shall warrant structural damages and thus, plastic hinge degradation. The scaling level at which 0.5% interstorey drift is reached is recorded as the intensity measure in IDA. The second criterion applies when all available solution algorithms in OpenSees fail to converge. The scaling level at which the convergence failure occurs is recorded as the intensity measure in IDA, which in turn generates the fragility curves. Since the recorded scaling level may not necessarily correspond to an interstorey drift threshold violation for most of the long duration subduction ground motions, the mean and standard deviation calculated using these scaling levels will lead to those non-parallel fragility curves. This observation is evident in Figure 31 (a) and (b) where a comparison of fragility curve is made while the degradation parameter varies.

6.3 Implications of Study Results

With the “what” component of the effects of long duration ground motion answered, the next logical question is “how” to account for these effects. Following the performance based design philosophy, a structure shall achieve a particular performance level at a particular ground motion intensity. Though to answer the “how” question is beyond the scope of this thesis, some preliminary ideas are proposed in light of the current structural design requirements in the NBCC 2015.

6.3.1 Recommendations for Performance Based Design

When performing time history analysis, the designer shall pay extra attention to the ground motion selection process. The IDA procedure used in this thesis is rather time consuming and may not be suitable in a fast paced design office. Time history analysis, or the 1.0x scaling ground motion used in IDA, is prescribed in NBCC 2015 for irregular buildings meeting the definition in the publication. Currently NBCC 2015 requires a selection of eleven spectrally matching ground motions to gain an acceptable level of reliability in the analytical results.

The recommendation is two-fold. First, the uniform hazard values may be increased for region prone to subduction seismic hazard. Second, the eleven selected ground motions must include at least seven long duration records for site prone to subduction seismic event. The goal is to increase the seismic demand a structure must design for, so that the member sizes are reasonably large enough to allow for loss of capacity due to cyclic degradation.

6.3.2 Recommendations for Force Based Design

The equivalent static force procedure is most common for a regular building. The goal of the recommendation is similar to that for performance base design, i.e., to increase the seismic demand required for structural design. A load increasing factor may perhaps be easiest to implement to the force base design to account for long duration ground motion. Such a factor will be location dependent so that the subduction seismic hazard is implicitly accounted for. For example, a load increasing duration factor of 1.5 may be applied to Victoria, B.C., whereas a factor of 1.0 to the Saskatoon, S.K., where subduction earthquakes are unlikely.

The study in Chapter 4 may perhaps shed light on the magnitude of such load increasing factor. This factor can also implicitly include information from performance based design to the well adopted force based design. The plastic hinge defining parameters, θ_p and λ, are both related to the member size and reinforcement ratio. If a performance level, for instance probability of drift exceedance shall be 50% at 1.0x scaling intensity, is specified, then the two plastic hinge defining parameters can be varied to achieve the desirable fragility curve using the same procedure as this thesis. Using the statistic regression equations in the PEER Report 2007/03 publication, the design can then back calculated the required dimensions and reinforcement ratios, thus required capacity of the member. The load increasing factor will then be the ratio of this back calculated required capacity to the capacity obtained from the existing NBCC 2015 structural design procedure. The development of a load increasing factor map is expected to be a major research project and shall be carefully carried out.

6.3.3 Recommendations for Seismic Retrofit

A structural ductility assessment shall precede any seismic retrofit work. As pointed out in Section 5.4.1, adding ductility to an already ductile structure has little effect on closing the gap between the fragility curves for crustal and subduction ground motions. However, introduction of ductility to a brittle structure will significantly enhance its fragility curves under long duration ground motions. The use of dampers or isolation devices is out of the scope of this thesis.

Chapter 7: Conclusion

In general, a building is more likely to experience drift exceedance if shaken by a long duration ground motion. This observation is repeatedly seen from the reinforced concrete shear wall and the frame structure investigated in this thesis, and is most evident when the building's ductility is moderate. At the same ground motion intensity, the probability of drift exceedance is usually higher under long duration subduction ground motion than under its short duration crustal counterpart. Buildings in subduction regions around the globe, including the Cascadia Subduction Zone, may be susceptible to large and long duration seismic events. This study has demonstrated that many older buildings would be at a high risk if such an event were to occur.

Though seismicity is usually considered in the design of modern buildings, effect of the ground motion duration is not explicitly accounted for. Results in these studies have shown that some discrepancies between the probability of drift exceedance of crustal and subduction ground motions do exist and that the effect of the duration of motions may be important in structural design and assessment. Structural ductility and the amount of material cyclic degradation have been shown to significantly affect fragility curve results.

7.1 Future Research Needs

Structural system investigated in this thesis is limited only to reinforced concrete construction. Similar fragility curve study shall be carried out for steel, masonry, and concrete shear wall buildings to gain better understanding of the effects of long duration ground motion. The reinforced concrete frame system used in this thesis, though a very common building type in the Washington State of USA, it is not the predominant building type in the Province of British Columbia in Canada. Computer models and hysteretic bahaviour shall be revised for the more common concrete shear wall construction.

Long duration ground motions are not in abundance. By varying the analysis parameters, such as the number of input ground motions or the increment of scaling level, more statistically significant results may be achievable. Field testing on a shake table may perhaps help to confirm findings in this thesis, and to quantify benefits resulting from the proposed recommendations.

References

ATC 72-1 (2010). *Modeling and acceptance criteria for seismic design and analysis of tall buildings*. Applied Technology Council, Redwood City, CA, October 2010.

ATC 78-1 (2012). *Evaluation of the methodology to select and prioritize collapse indicators in older concrete buildings.* Applied Technology Council, Redwood City, CA, 2012.

Bebamzadeh, A., Ventura, C.V., Fairhurst, M. (2015). *S2GM: Ground motion selection and scaling database*. Annual Los Angeles Tall Building Design Council Meeting, Los Angeles, CA.

Birely, A.C. (2012). *Seismic Performance of Slender Reinforced Concrete Structural Walls*. Ph.D Thesis Dissertation, University of Washington, Seattle, WA.

Chandramohan, R., Baker, J.W., and Deierlein, G.G. (2016). *Impact of hazard-consistent ground motion duration in structural collapse risk assessment*. Earthquake Engineering and Structural Dynamics: In-Press.

Chandramohan, R., Baker, J.W., and Deierlein, G.G. (2014a). *Quantifying the influence of ground motion duration on structural collapse capacity using spectrally equivalent records*. Earthquake Spectra In-Press.

Chandramohan, R., Baker, J.W., and Deierlein, G.G. (2014b). *Digital Appendix to Quantifying the influence of ground motion duration on structural collapse capacity using spectrally equivalent records*. Earthquake Spectra.

Hancock, J., and Bommer, J. J. (2006). *A state-of-knowledge review of the influence of strong-motion duration on structural damage.* Earthquake Spectra: August 2006, Vol.22, No.3, pp. 826-845.

Hancock, J., and Bommer, J. J. (2007). *Using spectral matched records to explore the influence of strong-motion duration on inelastic structural response.* Soil Dynamics and Earthquake Engineering, April 2007, Vol. 27, pp. 291-299.

Haselton, C.B., Liel, A.B., Lange, S.T., and Deierlein, G.G. (2008). Beam-Column Element Model Calibrated for Predicting Flexural Response Leading to Global Collapse of RC Frame Buildings. PEER 2007/03 Report. Pacific Earthquake Engineering Research Center, Berkerly, CA, May 2008.

Ibarra, L.F., Medina, R.A., and Krawinkler, H. (2005). *Hysteretic models that incorporate strength and stiffness deterioration*. Earthquake Engineering and Structural Dynamics, Vol. 34, pp. 1489-1511.

Lignos, D.G. and Krawinkler, H. (2012). Sidesway collapse of deteriorating structural systems under seismic excitations. Rep.No.TB 177, The John A. Blume Earthquake Engineering Research Center, Stanford University, Stanford, CA.

Open System for Earthquake Engineering Simulation (OpenSees) (2014). Pacific Earthquake Engineering Research Center, University of California, Berkeley. Retrieved June 15, 2014 from the World Wide Web: http://opensees.berkeley.edu/

Raghunandan, M. (2013). *Influence of Long Duration Ground Shaking on Collapse of Reinforced Concrete Structures* (Doctoral dissertation). University of Colorado, Boulder, CO.

Raghunandan, M., and Liel, A.B. (2013). *Effect of ground motion duration on earthquake-induced structural collapse*. Structural Safety, Vol.41, pp. 119-133.

Raghunandan, M., Liel, A.B., and Luco, N. (2015). *Collapse risk of buildings in the Pacific Northwest Region due to subduction earthquakes*. Earthquake Spectra: November 2015, Vol. 31, No.4, pp. 2087-2115.

U.S. National Park Service. (2014). Geological Illustrtions. Retrieved August 18, 2014 from the World Wide Web: http://www.nature.nps.gov/geology/education/education_graphics.cfm

Vamvatsikos D, Cornell CA (2002). *Incremental dynamic analysis*. Earthquake Engineering Structural Dynamics 2002;31(3):491–514.