Fragility Assessment of High-Rise Reinforced Concrete Buildings

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FRAGILITY ASSESSMENT OF HIGH-RISE REINFORCED CONCRETE BUILDINGS

by

Hezha Sadraddin

A thesis submitted to the Graduate College
in partial fulfillment of the requirements
for the degree of Master of Science in Engineering (Civil)
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This research presents a probabilistic seismic responses analysis of high-rise reinforced concrete (RC) buildings using fragility assessment method. Three RC buildings, having the same plan dimension and height (12 story each) but different in structural configurations, were designed and their seismic responses were compared. First building is a Moment Resisting Frame (MRF), second is a MRF with exterior shear walls, and the third building consists most shear walls. Buildings were designed for high-seismic activity zone (i.e. Los Anglos) using the Equivalent Lateral Force (ELF) for seismic loading calculation. Sixteen real ground motion pairs were selected and scaled, then applied orthogonally to the buildings to perform the Incremental Dynamic Analysis (IDA). Fragility curves were developed based on the IDA results for the three limit states including slight damage, moderate damage, and collapse to show the probabilistic comparison of seismic responses among the three buildings in both $x$ and $y$-directions. It was observed from the fragility assessment results that generally shear walls improve buildings’ seismic performance. However shear wall configuration also affects the seismic performance which needs further study.
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I would also like to express my appreciation to my family members and all friends in Kurdistan and United States. They helped me a lot to complete my degree.

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Hezha Sadreddin
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1 Introduction

1.1 Background

Developing mega cities leads to increase population in the city and there are not sufficient spaces provided by large number of buildings to accommodate the increasing population. High-rise buildings address this challenge as one of the solutions for the developing countries and mega cities. In addition, high-rise buildings give aesthetic to cities and they are signs of modern development. Comparatively, lower high-rise buildings (approximately 8~20 stories) are more common than super high-rise buildings (usually more than 30 stories) over the worlds. Although it is imperative to know structural behavior of both buildings including their seismic performances. High-rise buildings exhibit far more complex dynamic properties that require careful study and a complete understanding before they can be confidently resided in.

Reinforced Concrete (RC) is a common building material which have been used to construct high-rise buildings for several decades. Different RC building shapes can be achieved using advanced molds. Shear walls are used in RC buildings to increase their resistance against all kinds of gravity and lateral loads (including seismic loading) they may experience during their life span. Placing of shear wall at the optimal position in the buildings is essential to achieve sustainable and resilient building performance under both daily and extreme load conditions.

Earthquakes are one of the most hazardous natural disasters that attacks human and cause large damages especially in regions where defined as high-seismic zone by geologists. West coast side of the United States was defined as a high-seismic zone and a number of earthquakes attacked that region such as Northridge earthquake in 1994 which
hit Los Angeles, California that caused 57 deaths, 5000 injuries, and $20 billion losses in that area. Therefore, designing and analyzing structures to resist seismic attack is essential in those regions not only for new buildings, but also for existing buildings.

Various seismic analysis approaches were proposed including both static and dynamic methods. Although seismology has been continuously advancing during the century, it is impossible to predict future earthquakes’ severity and time of attacking. Therefore previous earthquake data are still widely used to analyze buildings resulting in robust buildings for future earthquakes.

Due to the difficulties in predicting earthquakes and its random nature, probabilistic analysis was proposed in analyzing building seismic responses. In addition to uncertainties in seismic loads, uncertainties associated with building material, design process, building geometry, and construction will also lead to the use of probability to predict building responses. Fragility curve assessment is one of the probabilistic methods which shows the conditional probability of exceeding a certain damage level. Fragility assessment has been widely adopted in earthquake engineering to understand the seismic performance of different buildings.

1.2 Objectives and Scope

The goal of this research is to understand the effects of shear walls on the seismic performance of high-rise RC buildings designed according to code. To achieve this research goal, three high-rise RC residential buildings, each 12 story high and having the same plan dimensions with different structural configurations, were designed and analyzed and their seismic responses are compared using a probabilistic method. The chosen RC buildings’ configurations are Moment-Resisting Frame (MRF), MRF with exterior shear...
walls, and MRF with both exterior and interior shear walls. Three buildings were analyzed using Incremental Dynamic Analysis (IDA) and buildings performances for three limit states including slight damage, moderate damage, and collapse were determined from IDA curves in terms of spectral acceleration at fundamental period with 5% damping. Then, probabilistic analysis was performed using fragility assessment method. To perform the fragility assessment, fragility curves were developed for the three limit states and fragility curves of the three buildings in the three limit states were compared to determine the best configuration of high-rise RC buildings. Finally, linear regression analysis was performed to show the effect of amount of shear walls on seismic performances.

1.3 Thesis Organization

This research contains seven sections which explain the seismic responses of three high-rise RC buildings. Three buildings designed by performing static approximate method (Equivalent Lateral Force, ELF) and analyzed using dynamic method (IDA). Three limit states were determined from IDA curves and fragility curves were developed for the three buildings in both x- and y-direction of three limit states.

Chapter one discusses the general background of this research including high-rise RC buildings and seismic effects on the buildings. Also, this chapter lays out the main objectives of the research and thesis outlines.

Chapter two explains basics of multi-story buildings and how to differentiate high-rise buildings from low-rise and mid-rise buildings. High-rise RC building configurations and their seismic performance are explained. The chapter then discusses the seismic analysis approaches, fragility curve types and IDA in details. Finally, chapter shows pervious works done by researchers on fragility curves of RC buildings.
Chapter three discusses the description and RC design of three RC buildings. In the beginning, this chapter describes the lay out of the three sample buildings under investigation by showing their plan and elevation views. Then the RC design process including the detailed procedure of Equivalent Lateral Force (ELF) method is explained. Finally, all the design results are shown and listed.

Chapter four explains the nonlinear modelling of the three RC buildings for numerical simulation using a software named SeismoStruct to obtain accurate model representing real buildings. Geometric and material nonlinearities and material types (i.e. concrete and reinforcement) used in this research are explained. This chapter presents the building members cross-section formulation. Finally, slab modelling using rigid diaphragm are explained.

Chapter five focuses on the dynamic analysis of the building models established in Chapter 5. Firstly, modal analysis results are shown including the modal periods and shapes to validate numerical models established in SeismoStruct and to obtain fundamental periods which were used to select and scale ground motions for the nonlinear time history analysis. IDA process that utilizes the results from the time history analysis is discussed next. Finally, this chapter derives the fragility curves formula and presents the procedure to create fragility curves.

Chapter six discusses the results from the IDA and fragility curves in great details to show and compare seismic responses of the three sample buildings. In addition, method for determining the limit states is explained. Fragility curves of different limit states were compared among the three building models to determine the best high-rise RC buildings’ configuration subjected to strong ground motions in all selected limit states. Finally,
regression analysis was performed to show the relationship between the amount of shear wall in the buildings and the mean of the observed intensity measure from the IDA curves.

Chapter seven concludes the observations and outcomes from this research and provides future recommendation.
2 Background and Literature Review

2.1 Introduction

This chapter discusses background and literature of fragility assessment of Reinforced Concrete (RC) buildings. Differences between three (low, mid, and high-rise) RC buildings were explained. In addition, common configurations of high-rise buildings were defined which are Moment Resisting Frame (MRF), MRF with shear walls, core and outrigger systems, tubular RC systems, and hybrid systems. For each high-rise RC building configuration, its proper number of stories are listed for resisting lateral loads and remaining stable. Seismic analysis method are briefly discussed including both dynamic and static approaches. The definition and categories of fragility curves are explained. Details of Incremental Dynamic Analysis (IDA) including its definition terms, IDA curves properties, and its usage in performance evaluation are presented in this chapter. In the end of this chapter, literature review of fragility assessment of RC buildings are discussed including RC building types and numbers, number of ground motions, seismic analysis approaches, analysis software, and limit states they used in their respective research.

2.2 Multi-story Buildings

Multi-story buildings are defined as buildings which have more than one story and they are classified into three classes based on their heights; namely low-rise, mid-rise, and high-rise buildings. However, the exact height limit of each class is still in debate and different researchers have different definition of height limits for the three types of buildings. Maximum story number of low-rise buildings is three according to Embark (2008). Low-rise buildings are most common buildings for multi-family houses especially in the metropolitan areas with high dense populations. Building is considered as a mid-rise
building if it has four or more stories but less height compared to those defined for high-rise buildings.

According to Ji et al (2007), buildings are considered as high-rise when their heights are more than 35 meters (114.83 feet) or twelve stories. HAZUS MR4 (2004) technical manual, which is developed by Federal Emergency Management Agency (FEMA) and is used to predict losses due to earthquake, considers eight story or more as high-rise buildings. The Council on Tall Buildings and Urban Habitats (CTBUH), an international council, defines high-rise buildings if their heights are more than 50 meters (165 feet) or they have more than fourteen stories. In addition, other building factors are considered to define a high-rise buildings as explained bellow:

1. Height relative to context and building location: Fourteen-story building may not be considered as tall building if its height is much lower than other adjacent buildings such as those located in the downtown areas of Chicago or Hong Kong. On the other hand, in the cities which have low-rise buildings twelve-story buildings are considered as high-rise building such as European cities.

2. Proportions: Slender buildings are considered as high-rise buildings, however, they do not have enough height. In contrast, fourteen-story buildings are not considered as high-rise building because if their height to width ratio is small.

3. Structural configuration: When braces are used to resist wind and seismic loads, buildings may be classified as high-rise buildings.

4. High-rise building technologies: Buildings contain special high-rise technologies, such as elevator, fire protection system, and waste disposal system may classified as high-rise building.
2.2.1 Reinforced Concrete Structures

Development in concrete technology in the last century and the advantages of RC buildings encourage designer and owner to focus on RC multi-story buildings. RC buildings cost less compared to steel structures with similar dimensions, they are easier to construct and they have more resistance to fire. In addition, RC buildings have more mass than steel or timber buildings which make them more stable to resist wind and seismic loads. Because stiffer structures and higher damping minimize the motion perception. The highest available concrete compressive strength in the markets is 165.5 mpa (24 ksi). Existing concrete available at different compressive strengths, construction techniques, building configurations, and structural analysis development helps to construct highest buildings with RC materials. Examples of such RC high-rise buildings include Petronas towers (452 meters high), Jin Mao (421 meters high) and Burj Khalifa (more than 800 meters high) (Rizk, 2010)

2.2.2 Reinforced Concrete Building Configurations for Lateral Force

Researchers proposed different configurations for RC buildings especially for high-rise buildings. RC building configurations generally consist of different structural elements such as beam, column and shear wall with different forms to achieve effective lateral load resistance against wind and seismic forces. Ji et al (2007) classified RC buildings into the following types.

2.2.2.1 Moment Resisting Frame (MRF)

Moment resisting frames consist of traditional beams and columns without shear walls. MRF can be used to resist both gravity loads and lateral loads. The floor slab serves as a horizontal diaphragm to transfer vertical loads onto beams and columns.
2.2.2.2 Moment Resisting Frame with Shear Walls

Moment resisting frame with shear walls is similar to first type except shear walls added to the MRF to increase lateral-load resistance because shear walls make buildings stiffer especially for lower high-rise buildings. Shear wall decreases lateral sway when building subjected to lateral forces such as earthquake and wind loads.

2.2.2.3 Core and Outrigger System (COS)

Similar to shear wall system, this type of RC building configuration is a combination of shear walls with beams and columns in a different form which is extensively used in high-rise buildings. COS consists of a central shear wall core which is connected to other parts of the frame such as beam and column or flat slab system and lateral loads are transmitted by floor diaphragms from exterior part of frame to the central core. The exterior beam-column or flat plate system is known as Outrigger (Ji et al 2007).

2.2.2.4 Tubular RC System

Tubular buildings are designed to act as three dimensional object to resist overturning moment. This system has the same structural components as previous types such as shear wall and beam-column frame. Tubular system, however, differs from other configurations by having a tube construction such as tube-in-tube, bundled tube, trussed tube, framed or braced tube to achieve different functions. For example, bundled tube is used in large buildings to reduce exposed surface area to the wind load and tube-in-tube system consists of two shear wall tubes which are located inside each other. These two tubes are connected to act as one object to resist lateral loads (Ji et al 2007).
2.2.2.5 Hybrid System (HS)

HS is a complex high-rise building, but they are more efficient because they adopt both concrete and structural steel components in the form of a composite components to take the advantages from both materials’ properties and construction techniques. These systems are a combination of concrete and steel system (i.e. composite structure). Best examples for HS are Petronas Tower (1483 feet high) and Jim Mao building (1380 feet high) which are located in Kuala Lumpur, Malaysia and Shanghai, China, respectively. First one used high-strength concrete material in core wall, column and ring beams and slabs, but slabs and slab beams constructed from steel. Tower core wall connected to the outer columns in order to provide more stiffness to resist lateral loads. Second building used steel trusses to connect interior core wall to the outer mega-columns (Ji et al 2007).

2.2.3 Seismic Performance of RC Buildings

Seismic performance of RC buildings mostly depends on the building configurations and the number of stories of the buildings. There are many factors affecting seismic performance of the buildings such as building stiffness, strength, and ductility. Buildings’ stiffness can be increased by adding shear walls. Also, shear walls decrease lateral sway of buildings which gives more stability to the buildings. Different RC shear walls configurations are adopted to increase seismic resistance such as tubular system which is more common for high-rise buildings as explained before.

Another factor which is important to be considered in RC buildings is ductility because concrete is a brittle material. Ductility can be increased in RC buildings by using special details in reinforcement. Each building configuration discussed in Section 2.2 has
a applicable range of the story numbers as shown in Table 2-1. Including their respective ductility and stiffness levels.

<table>
<thead>
<tr>
<th>RC Building Configuration</th>
<th>Number of stories</th>
<th>Stiffness</th>
<th>Ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>MRF</td>
<td>15-20</td>
<td>Low</td>
<td>High</td>
</tr>
<tr>
<td>Braced Frame</td>
<td>20-30</td>
<td>High</td>
<td>Low to moderate</td>
</tr>
<tr>
<td>Shear Wall Buildings</td>
<td>25-30</td>
<td>High</td>
<td>Low to moderate</td>
</tr>
<tr>
<td>HS</td>
<td>30-40</td>
<td>High</td>
<td>Moderate to high</td>
</tr>
<tr>
<td>COS</td>
<td>50-60</td>
<td>High</td>
<td>Low to moderate</td>
</tr>
<tr>
<td>Framed Tube System</td>
<td>60-70</td>
<td>High</td>
<td>Moderate to high</td>
</tr>
<tr>
<td>Tube-in-Tube System</td>
<td>70-80</td>
<td>High</td>
<td>Moderate to high</td>
</tr>
<tr>
<td>Trussed Tube System</td>
<td>80-100</td>
<td>High</td>
<td>Moderate to high</td>
</tr>
<tr>
<td>Bundle Tube System</td>
<td>120-150</td>
<td>High</td>
<td>Moderate to high</td>
</tr>
</tbody>
</table>

2.3 Seismic Analysis Approaches

Different seismic analysis approaches have been developed to predict buildings responses when subjected to strong earthquakes. With the development of computer software, analysis methods are expanded from static to dynamic and from linear to nonlinear analysis to obtain more realistic seismic response of buildings. There are three static seismic analysis methods including Equivalent Lateral Force (ELF), the Conventional Pushover and Adaptive Pushover. Dynamic methods consist of Multi-modal spectral, Nonlinear Time history and Incremental Dynamic Analysis (IDA).
Researchers showed that all these methods cannot analyze all buildings except for the detailed non-linear dynamic analysis (Shah and Tande 2014). For instance, ELF is only suitable for determine seismic forces of regular buildings up to 90 m high and located in seismic zone I and II, while dynamic analyses such as nonlinear time history analysis can be applied to both regular and irregular buildings in the seismic zone IV and V (Bagheri 2012).

In this research, Incremental Dynamic Analysis (IDA) was adopted which uses a number of non-linear dynamic analysis. This method subjects building models to natural ground motions to result in more realistic responses for use in probabilistic assessment. Probabilistic assessment is necessary for seismic response analysis because of its uncertainties especially ground motion uncertainty. In this study, fragility assessment, one of the probabilistic method, was adopted by creating fragility curves to show seismic performance of different RC high-rise buildings. In addition, fragility curves can compare buildings easily in different limit states.

### 2.3.1 Fragility Curve

Fragility curve is defined as the conditional probability which exceeds a specified limit state and evaluates seismic vulnerability of the structure. Fragility curve shows the probability of structure damage as a function of ground motion intensity measure (IM) such as Peak Ground Acceleration (PGA), spectral acceleration at the fundamental building period with 5% damping $S_a(T_f, 5\%)$ or any other intensity measures. Figure 2-1 shows an example of a fragility curve. Fragility curve can be expressed as:

$$f_{DS}(IM) = P(DS|IM) \quad \text{Eq. 2-1}$$
where,

\[ IM = \text{the ground motion intensity measure.} \]

\[ DS = \text{the damage state.} \]

\[ P = \text{the probability of exceeding a damage level.} \]

**Figure 2-1** Fragility curve of collapse limit state shows the way of determining the probability of 50% of collapse.

Figure 2-1 shows the probability of collapse limit state and the probability from 0% to 100% of collapse can be determined from the curve. For example, if we want to determine 50% probability of collapse, we shall determine the point on the fragility curve which has a vertical axis value equal to 0.5. Then, value on the horizontal axis representing
the ground motion intensity (i.e. IM = 0.14g) is determined which corresponds to the probability of 50% of collapse.

Different methodologies were developed to show fragility relationship between IM and the building responses. These methodologies are classified into four types which are experiential, analytical, empirical, and hybrid fragility curves (AmiriHormozak 2013). Detail of each type is explained in the following sections.

2.3.1.1 Experiential Fragility Curve

The experiential fragility curves are developed based on the opinion or expert data collected from people when there was not sufficient data to predict the probability of the building damage due to earthquake. Experiential fragility curve was first developed by the Applied Technology Council (ATC) in 1985. ATC collected data by taking surveys with experts to predict seismic response. In that survey, ATC determined the possibility of 7 level of damage on a bridges in terms of Modified-Mercalli Intensity (MMI). Then, the survey results were analyzed to create damage curves. Results obtaining from this method are subjective because they are completely depend on experts’ experiences, number of experts participated in the survey and their judgment (AmiriHormozak 2013).

2.3.1.2 Empirical Fragility Curve

Empirical fragility curves are constructed based on field observations. The process for developing an empirical fragility curve starts after an earthquake and then damage data are collected. With these data and shake map which shows ground motion intensity distribution, damage probability matrices can be developed (AmiriHormozak 2013). However, this method has the following disadvantages:
1. Observers are not able to collect data for different types of buildings and damage states.

2. For the same building, various damage states are defined because each observer’s definition is different.

3. People generate various shake map for same earthquake event.

### 2.3.1.3 Analytical Fragility Curve

Analytical fragility curve is more common than the other curves which uses numerical simulation to predict damage distributions. For complex buildings, analytical method is adopted to create fragility curves. To yield accurate results, building models are usually calibrated against experimental data and modified based on experts’ judgment, observations and previous researches. Seismic analysis methods are used to develop the analytical curve such as elastic spectral analysis, static pushover analysis, nonlinear dynamic analysis, and so on (AmiriHormozak 2013).

### 2.3.1.4 Hybrid Fragility Curve

Hybrid fragility curve is developed based on a combination of experiential, empirical and analytical method and it gives more realistic fragility curve. Since the limitations and insufficient data of the empirical and experiential method and the complexity of building modelling in the analytical method, hybrid method was proposed when there are not sufficient damage data at a specified intensity level for a geographical area (AmiriHormozak 2013).

In this study, the analytical fragility curve method was used to develop fragility curve using IDA as discussed next.
2.3.2 Incremental Dynamic Analysis

Bertero firstly proposed the idea of incremental dynamic analysis (IDA) in 1977 and it has been subjected to substantial development by many researchers at the end of last century and the beginning of this century. This analysis method was adopted by the Federal Emergency Management Agency (FEMA 2000a) and is considered as the state-of-the-art method to estimate the structural responses under seismic loadings. IDA is a parametric analysis which predicts complete structural responses and performances. In this analysis, a properly defined structural model is subjected to a suite of ground motion records and the intensity of these ground motions are gradually increased using scale factors. The intensity continues to increase when the whole structural responses range from elastic to the nonlinear followed by structural collapse (Vamvatsikos 2002). In the end, a number of curves depicting the parameterized responses versus the ground motion intensity levels are produced. IDA performs a huge number of non-linear time history analysis. For example, a complete IDA may have 20 or more ground motion pairs and each is scaled to 12 levels leading to $20 \times 12 = 240$ times non-linear time history analyses. Although it takes a long time to perform IDA, it can provide the whole range of structural responses from elastic to collapse. With the development of computing technology every day, software were created to perform the IDA making it possible for both practical and research purposes.

2.3.2.1 Terminology in IDA

Vamvatsikos and Cornell (2005) defined common terms in the IDA as listed below:

- Scale factor: a positive scalar which multiplies to ground motion to increase the intensity. Scale factor can be increased in a constant steps or distinct steps.
- **Intensity Measure (IM):** a positive scalar which depends on the unscaled ground motions and it is increased monotonically with scale factor. IM can be increased by multiplying the scale factor to the ground motion.

- **Damage Measure (DM):** a positive scalar which is also known as a Structural State Variable. DM characterizes more structural response which is subjected to prescribed seismic load. Choosing an applicable DM depends on structure and the application of structure. Possible selection for the DM could be maximum base shear, node rotations, peak story ductility, various proposed damage indices such as global cumulative hysteretic energy, a global Park-Ang index, peak roof drift $\theta_{\text{roof}}$, the floor peak inter-story drift angle for all story of the building or the maximum inter-story drift angle.

- **IDA curve:** a graph of DM versus IM. IDA curve can be plotted in two or more dimensions relying on the IM and at least one of them must be scalable.

- **Single-Record IDA curve:** also known as IDA curve or Dynamic Pushover (DPO) curve (Vamvatsikos and Cornell 2005). As mentioned in the IDA introduction, single-record IDA is obtained by applying a number of non-linear time history analysis for the same record with different scale factors. The intensity of the ground motion incrementally increased in each non-linear time history analysis by multiply the amplitude of ground motion to the incremented scale factor. From these time history analysis results, DM is recorded (i.e. maximum inter-story drift). A curve relating the DM value to the IM is obtained and this curve is known as a single-record IDA curve. Using single-record curves is not enough to estimate the response of the structure and display the effects of future earthquakes (Yu et al
Single-record IDA curve helps researchers and engineers to know the response of the structure under different intensities for a single earthquake (Kruep 2007). With single IDA curve, value of IM and DM can be determined.

- Multi-Record IDA curve: Since single-record IDA curve cannot capture building seismic responses to future earthquakes, multi-record IDA curve is used to obtain a better prediction of the building response. The multi-record IDA curve is therefore a collection of single-record IDA curves for a single building obtained from different ground motions, which are all parameterized on the same IM and DM (Vamvatsikos and Cornell, 2005). It is usually difficult to construct a structure to resist all ground motions, but creating multi-record IDA curves with the same scaling parameter for different ground motions will reduce the probability of building damages under future earthquakes. Ground motion selection is an important step to create multi-record IDA curves and quite a number of records are needed to capture the entire response range.

### 2.3.2.2 IDA General Properties

IDA curve visualizes the structural responses and shows structural behavior subjected to ground motions. Buildings have different IDA curve shapes depending on their capacities (i.e. strength, stiffness, ductility) to resist seismic loads. In addition, researchers choosing different IM and DM values based on their research objectives, will resulting in different IDA curves. For example, IDA curve using base shear as the DM is different from the curve using the maximum inter-story drift as the DM.

Slope of the IDA curve is a main indicator to understand the behavior of the structure. The structure has elastic response when the slope of the IDA curve is linear which
means that the proportion of the DM is linear with the IM. When the IDA curve becomes non-linear representing the structure enters into the nonlinear range as shown in the Figure 2-2 (Chan 2005). Generally, when the scale factor is low, IDA curve is a straight line which means that the structure is in the elastic region. When the scale factor becomes higher, IDA curve starts to bend meaning that the structure is close to yield. Building is considered as collapse when curve become flat line.

The non-linear part is terminated to four different types according to their corresponding IM as defined by Vamvatsikos and Cornell (2005) and the shapes for all four types are shown in the Figure 2-3. Curve (a): the curve sharply softens when the initial buckling completed and the structure has a big drift which causes the structure to collapse. Curve (b) has a small hardening in its non-linear region comparing to the other two curves (c) and (d), which harden and weave around the elastic region meaning that the global

![Diagram of IDA curve showing linear and non-linear regions](image)

**Figure 2-2 Linear and non-linear region on the IDA curve.**
displacement of the inelastic range is very close or equal to the displacement of the elastic model (Vamvatsikos and Cornell, 2005).

![Graph showing four different behaviors of IDA curves](image)

**Figure 2-3 Four different behavior of IDA curves (Vamvatsikos and Cornell, 2005).**

Softening case means building collapses at smaller value IM and it has larger DM i.e. maximum inter-story drift. In contrast, hardening means that IDA curve in the nonlinear region weaving which means DM value increased and decreased by increasing IM. Collapse of the IDA curves having hardening property is calculated from end part of the curve which become flat line. Finally, the IM values at collapse and different damage
values indicates the seismic capacity of a building model. For example, curve (a) has the lowest value of IM and curve (d) has the highest value of IM among the four curves shown in Figure 2-3.

### 2.3.2.3 Performance Evaluation for IDA Curves

Performance level or limit state of the IDA curves is an important part in assessing building seismic response. Buildings are usually evaluated at the limit states of the IDA curves and the fragility curves are constructed based on those limit states. Limit states are defined as collapse, immediate occupancy or other limit states depending on the performance type. For instance, global collapse can be determined based on the IM and DM values when one observes the dynamic instability, while immediate occupancy performance level is depended on a specified DM, usually the maximum inter-story drift. Issues may arise when defining the performance level based on DM values. In some IDA curves vertical line of the DM crosses more than one points. Researchers shall be careful to choose the correct point for the limit state on the IDA curve using the DM based rule and/or the IM based rule.

The idea of the DM-based rule is that the limit state exceeded when the IDA curve reaches a predefined value of DM (Vamvatsikos and Cornell, 2005). For example, if the collapse is defined when maximum drift ratio reaches 10%, the portion of IDA curve greater than 10% is considered as collapse (see Figure 2-4). The advantage of the DM-based rule is simple and easy. In addition, DM-based rule is accurate in estimating the performance level of the building. However the DM-based rule requires the structural modeling with a high precise level to obtain accurate structural responses during dynamic simulation (Chan 2005).
Figure 2-4 DM-Based rule uses maximum inter-story drift to define capacity point (Vamvatsikos and Cornell, 2005)

The IM-based rule is also widely used in determining the limit state on the IDA curves. According to the IM-based rule, an IDA curve is divided into two parts which are known as collapse and non-collapse region (see Figure 2-5). The collapse area is the upper part of the IDA curve, while the non-collapse region is the lower part of the IDA curve. Vamvatsikos (2002) pointed out that it is difficult to determine the separation point between these two regions. According to FEMA 2000a, capacity point is the point of collapse and non-collapse separation and this point is the last point on the IDA curve with
a slope equal to 20% of the elastic slope. The IM-based rule is better than the DM-based rule to assess structural collapse (Chan 2005).

![Graph showing IM-based rule](image)

**Figure 2-5 IM-based rule uses 20% slope rule to define capacity point**

(Vamvatsikos and Cornell, 2005)

### 2.4 Literature Review

Researchers have been developing analysis methods to show the probability of certain damage level using fragility curves when RC buildings subject to earthquakes. Previous researches have been focused on low and mid-rise RC buildings including both MRF and shear wall RC buildings. Due to the difficulties in analyzing high-rise RC buildings, only a few researches have been reported on the fragility assessments for high-
rise RC buildings. Each research used different number of ground motions to generate the fragility curves depending on the project objective. Fragility curves of different damage levels were developed and different codes were used to determine the damage levels such as FEMA 356, HAZUS MR4 (2004) earthquake technical manual, American Society of Civil Engineers (ASCE), and so on. Summary of previous researches on the fragility assessment of RC buildings including low mid and high-rise buildings is shown in Table 2-2 and detailed discussion is provided below.
Table 2-2 Shows summary of previous work

<table>
<thead>
<tr>
<th>Researchers</th>
<th>No. and type of RC buildings</th>
<th>Ground motion</th>
<th>Method to develop fragility curve</th>
<th>Computer program used</th>
<th>Limit states used to develop fragility curves</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dumova-Jovanoska (2000)</td>
<td>One mid and one high rise, six and sixteen stories</td>
<td>240 synthetic time histories</td>
<td>Non-linear time history analysis</td>
<td>IDRAC-2D Version 4.0</td>
<td>None damage, minor, moderate, severe and collapse</td>
<td>Time history analysis showed that damage shall not happen under seismic attack of designed intensity with 50% probability; obtained fragility curves of all limit states were consistent with other researchers</td>
</tr>
<tr>
<td>Hsieh et al (2013)</td>
<td>Two (1 and 3 stories) low and two (4 and 7 stories) mid-rise</td>
<td>Chi-Chi earthquake</td>
<td>Grid-based cluster division was used, compared to district based method</td>
<td>N/A</td>
<td>Intermediate damage and Collapse</td>
<td>Grid-based method gives more stable and convergent fragility curves; the numerical results verified the proposed method in developing reasonable fragility curve</td>
</tr>
<tr>
<td>Jiang et al (2012)</td>
<td>Three (3,6, and 10 stories) MRF considering as low, mid, and high-rise</td>
<td>10 ground motion pairs</td>
<td>Non-linear time history analysis</td>
<td>OpenSees</td>
<td>Fully operational, Operational, Repairable and Collapse prevention</td>
<td>Buildings designed based on current Chinese seismic code have good reliability against seismic forces</td>
</tr>
</tbody>
</table>
Table 2-2 Continued

<table>
<thead>
<tr>
<th>Researchers</th>
<th>No. and type of RC buildings</th>
<th>Ground motion</th>
<th>Method to develop fragility curve</th>
<th>Computer program used</th>
<th>Limit states used to develop fragility curves</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lagaros (2008)</td>
<td>Two groups: 1. Three 4 stories fully infilled, weak ground floor, and short columns. 2. One 3 stories designed for 6 behavior factor (q)</td>
<td>Nineteen</td>
<td>Non-linear static analysis (Pushover)</td>
<td>OpenSees</td>
<td>Slight, Moderate, Extensive, and Collapse</td>
<td>Comparing between two sets showed that the behavior of bare design gained when q=1 is the same as of fully infilled design gained when q=3.5</td>
</tr>
<tr>
<td>Syed (2013)</td>
<td>RC shear wall building for Nuclear Power Plant</td>
<td>Thirty ground motions</td>
<td>Non-linear time history analysis using Bayesian updating techniques</td>
<td>N/A</td>
<td>No-damage, minor damage, moderate damage, and significant</td>
<td>Bayesian updating techniques is a suitable tool to construct fragility curves and fragility developed based on drift ratio and maximum shear force were more conservative than maximum shear strain criteria.</td>
</tr>
<tr>
<td>Researchers</td>
<td>No. and type of RC buildings</td>
<td>Ground motion</td>
<td>Method to develop fragility curve</td>
<td>Computer program used</td>
<td>Limit states used to develop fragility curves</td>
<td>Results</td>
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<tr>
<td>Hernandez (2007)</td>
<td>One and two story RC buildings for both MRF and shear wall buildings</td>
<td>Five ground motions</td>
<td>Non-linear time history analysis</td>
<td>LARZ (Saiidi 1979)</td>
<td>Slight, Moderate, Extensive, and Collapse by HAZUS definition and minor, moderate, substantial, and major for Algan (1982)</td>
<td>For one story buildings fragility curves are almost the same for HAZUS and Algan limit states’ groups. For two story buildings shear wall buildings are more stable for all damage levels. Fragility curves for HAZUS and Algan limit states are same for all damage levels except complete damage.</td>
</tr>
<tr>
<td>Karapetrou et al (2015)</td>
<td>One high-rise (9 story) considering soil-structure interaction</td>
<td>Fifteen ground motions</td>
<td>IDA</td>
<td>OpenSees</td>
<td>Immediate occupancy and collapse prevention</td>
<td>According to the statistical results, soil-structure interaction and site effect have significant role on the seismic vulnerability and performance comparing with fixed base buildings.</td>
</tr>
<tr>
<td>Kircil &amp; Polat (2006)</td>
<td>3,5, and 7 story considered as low and med-rise buildings</td>
<td>Ten ground motions</td>
<td>IDA</td>
<td>IDRAC-2D Version 5.0</td>
<td>Yielding and collapse</td>
<td>According to the regression analysis, fragility parameters significantly changed with the number of stories.</td>
</tr>
</tbody>
</table>
2.4.1 RC Buildings

Researchers constructed fragility curves for various types of RC building including low, mid, and high-rise buildings and they employed different number of RC buildings based on the research objectives.

Dumova-Jovanoska (2000) developed fragility curves and damage probability matrices for mid and high-rise RC frame-wall buildings with six and sixteen stories, respectively located in Skopje (Macedonian). Both buildings designed according to Macedonian Design Code. Hsiesh et al (2013) developed fragility curves for two low-rise RC buildings which have one and three story and two mid-rise RC buildings which have four and seven story located in Taiwan. Jiang et al (2012) constructed fragility curves for three RC MRF buildings having three, six, and ten story considering as low, mid, and high-rise buildings. All buildings were designed according to the Chinese Code for seismic design. Each building was designed for four types of soil site conditions. Lagaros (2008) assessed two groups of low-rise RC buildings using fragility analysis. First group consists of three four story RC buildings which are, fully infilled, buildings with weak ground floor, and buildings with short columns. Second group is composed of a building with three story having six values of the behavioral factors (q) having magnitude one to six with one increment. Most current codes express the ability of buildings to absorb energy in inelastic deformation using behavior factor. Also, behavior factor used to decrease the seismic loads. Syed (2013) developed fragility curves for RC shear wall building used in nuclear power plants. Since experimental fragility curves are difficult to obtain for large scale RC shear wall buildings due to the high cost and impractical problems, hybrid fragility curves were adopted to evaluate seismic responses for box shaped RC shear walls. Hernandez (2007)
proposed fragility curves to evaluate seismic reliability for existing residential RC buildings in Puerto Rico which composed of both MRFs and shear walls buildings as requested by insurance companies to estimate their losses during disasters. Eighteen one story and eighteen two story MRF buildings were used. Also, eighteen one and two story shear wall buildings and 26 multi-story shear wall buildings having three to ten number of stories were used. Karapetrou et al (2015) developed fragility curves for high-rise (9 stories) RC MRF buildings considering soil-structure interactions and site effect under linear and nonlinear soil behavior. Results compared to buildings with fixed base assumption in order to evaluate effect of soil-structure interaction on the seismic performance and vulnerability. Kircil and Polat (2006) developed fragility curves for low-rise and mid-rise RC buildings consisting three stories, five stories, and seven stories in Istanbul, Turkey. These buildings were designed based on 1975 Turkish seismic design code.

Generally, selecting number of buildings depend on the research objective, for example, when low and mid-rise buildings are compared at least two buildings should be used. Also, if the objective of the research is to analyze a special building such as nuclear power plant, one building sample is enough. More building models were used when it is required. For example, Hernandez (2007) analyzed eighty existing buildings which was requested by insurance company.

2.4.2 Ground Motions

Researchers use a number of ground motions to analyze buildings including recorded previous ground motions and artificial ground motions. Since fragility curves show probability of damage limit states of buildings subjected to future earthquake,
selecting the number and specific ground motions is essential to create reliable fragility curves. Dumova-Jovanoska (2000) used 240 synthetic ground motions developed based on the Skopje (Macedonian) geological data because of the limited number of real earthquakes data in Skopje region. Hsieh et al (2013) observed damage recorded after the Chi-Chi earthquake in 1999 (i.e. one real ground motion was used) to develop empirical fragility curve for low and mid-rise buildings. Jiang et al (2012) used ten pairs of natural ground motions for each building frame. Lagaros (2008) used nineteen natural ground motion pairs including both rock and soil site conditions. Syed (2013) used thirty simulated ground motions. Hernandez (2007) used five ground motions, three of them were real recorded earthquake and other two ground motions were synthetic earthquake created based on the geological data of Puerto Rico. Karapetrou et al (2015) used fifteen real ground motions selected from the European Strong-Motion Database. Kircil and Polat (2006) used twelve artificial ground motions having magnitude of 7.5 and 40 second duration.

In summary, researchers used both real and synthetic ground motions. Synthetic ground motions have been used because real earthquakes are not available in those locations or existing ground motions may not have enough characteristics for analysis or design purposes. Also, some researchers need their own characteristics such as Kircil and Polat (2006) used twelve ground motions having magnitude of 7.5 and 40 second durations. Generally, ten to twenty ground motions were used depending on the analysis types. For example, in the IDA process, twelve and fifteen ground motions were used by Kircil and Polat (2006) and Karapetrou et al (2015), respectively.
2.4.3 Seismic Analysis Approaches

Different seismic analysis methods were utilized to obtain building responses subjected to severe earthquakes and to develop fragility curves such as non-linear time history analysis, IDA, pushover analysis, and so on. Dumova-Jovanoska (2000), Jiang et al (2012), Lagaros (2008), Hernandez (2007) used non-linear time history analysis and Syed (2013) used Bayesian updating techniques in performing non-linear time history analysis. Bayesian updating technique is a strong statistical method which used to combine previous or available data and comparing with existing data to establish better data such as updating the fragility curves in Syed (2013) research. Karapetrou et al (2015) and Kircil and Polat (2006) used IDA. Hsieh et al (2013) used grid-based cluster division, then compared to district-based method. Grid-based method is a method used to calculate number of damaged buildings within an area attacked by an earthquake if they assumed to be uniformly distributed over that region. This method divide an attacked area into small grid divisions then density of damaged buildings are calculated within each grid. District-based method is a common method used to investigate an area suffered from earthquake. This method uses district to calculate Peak Ground Acceleration (PGA) for buildings in order to develop fragility curves for buildings.

Therefore, non-linear time history analysis is the most common method in fragility assessment. Because nonlinear time history analysis is dynamic approach and it uses real ground motions. IDA rarely used to analyze buildings especially high-rise RC buildings because IDA requires a long time to analyze. Nevertheless, IDA is an accurate dynamic method and it shows the whole responses of the buildings from elastic to inelastic then collapse.
2.4.4 Computer Program


In summary, IDRAC-2D, OpenSees, and LARZ are software mainly for research purposes and have been used in dynamic analysis of buildings. The most common research software is OpenSees Research software is an open source software such as OpenSees, in which users can write their own program without permission from the original developer (i.e. license not required). Also, open source software can be redistributed and copied by all users. On the other hand, commercial software need license to use it and it cannot be developed or redistributed by the other users. Research software need programming background, but most commercial software are graphical inputs, which is easier for the users especially for those who are not familiar with writing programs. In this research SeismoStruct software was used which is a commercial software, but it is free for research purposes. Also, it is graphical software and it does not need any program or script to write which is easier for the users.

2.4.5 Limit States

Fragility curves are developed according to limit states of buildings. Researchers used different types of limit states depending on the research objectives. Dumova-

In summary, there are four limit states that are widely used which are slight damage, moderate damage, extensive damage, and collapse. Those researches which used IDA to obtain limit states for RC buildings used two limit states because of the difficulties in obtaining the maximum given drifts available in the seismic codes. Also, they used shapes of IDA curves to define limit states without comparing to seismic codes.
2.4.6 Research Results

Researchers used fragility curves for comparison of responses of different buildings and they obtained different results based on the research objective. Also, fragility curves were developed to show the probability of damage of buildings subjected to the future earthquakes such as Hernandez (2007) created fragility curves for insurance companies to expect their loss during disasters. According to the Dumova-Jovanoska (2000) results, nonlinear time history analysis showed that damage shall not happen under seismic attack of designed intensity with 50% probability and the fragility curves of all limit states were consistent with other researchers. Hsieh et al (2013) showed that grid-based method gives more stable and convergent fragility curves. In addition, the numerical results proofed that grid-based cluster division method can develop reasonable fragility curves. According to Jiang et al (2012), buildings designed based on the current Chinese seismic code have good reliability against seismic forces. According to the Lagaros (2008) comparison results, comparing between two sets of buildings shown in Table 2-2 showed that the behavior of bare design gained when behavior factor (q)=1 is the same as that of the fully infilled design gained when q=3.5. Syed (2013) showed that bayesian updating technique is a suitable tool to construct fragility curves and fragility developed based on drift ratios and maximum shear forces were more conservative than maximum shear strain criteria. Hernandez (2007) showed that one story buildings fragility curves are almost the same as those of both MRF and shear walls (i.e. seismic responses of the MRF and shear wall buildings are approximately similar), but for two story buildings, shear wall buildings are more stable at all damage levels. Also, Hernandez (2007) showed that fragility curves obtained based on the HAZUS limit states (2004) and the Algan limit states (1982) are the same for all damage
levels except for the complete damage limit state. Karapetrou et al (2015)’s fragility curves proofed that soil-structure interaction and the site effect have a significant role on the seismic vulnerability and performance comparing with fixed base buildings. According to Kircil and Polat (2006)’s regression analysis, fragility parameters, mean and standard deviation significantly changed with the number of stories. From the fragility curves and statistical analysis, Kircil and Polat (2006) determined maximum-inter story drift and spectral displacement limits for both immediate occupancy and collapse prevention.

In summary, most researches used fragility curves for comparison in probabilistic method between building types and number of stories, method to develop fragility curves, and analyzing building fragilities for special type of buildings. For example, Kircil and Polat (2006) used fragility curves to compare building responses with different number of stories and Hsieh et al (2013) used fragility curves to compare grid-based method and district-based method. Syed (2013) analyzed building fragilities of RC shear wall building used in nuclear power plants.

2.5 Conclusion

This chapter explains the general background and previous researches on seismic assessment of RC buildings. RC buildings are first classified based on buildings height including low-rise, mid-rise, and high-rise buildings. Then different configurations of high-rise RC buildings were described and their seismic performances were compared based on the number of stories by showing applicable height of each configuration. Seismic analysis approaches including static and dynamic methods were listed and briefly discussed among which IDA was adopted in this research. General introduction and terms of IDA were discussed and the IDA curve’s properties were explained. Fragility curves and the method
of creating fragility curves which are experiential, empirical, analytical, and hybrid methods were explained. In this research, analytical fragility curves were used. In the end of this chapter, previous works on fragility analysis of RC buildings was summarized to identify the building types and numbers, software, number of ground motions, limit states and seismic analysis method used to develop fragility curves.
3 Description and Design of Reinforced Concrete Buildings

3.1 Introduction

In this chapter, the design of three high-rise Reinforced Concrete (RC) buildings were described. These three buildings have the same plan dimensions and height (12 story) but different in lateral load resisting systems so the effect of different lateral load resisting systems on the seismic responses can be compared. The design of the buildings were carried out in STAAD Pro (2007) that follows American Society of Civil Engineers (ASCE 7) for load calculations and the Building Code Requirements for Structural Concrete and Commentary (ACI 318-05) code for RC member design. Equivalent Lateral Force (ELF) was used to calculate seismic loads which is a simple and approximate method used widely in practices. Buildings are assumed to be located in Los Angeles, California which is a high-seismic activity zone.

3.2 Buildings Description

Three high-rise RC buildings were designed and their seismic responses were assessed and compared. All the buildings have the same height (i.e. 12 stories height and 10 ft each) and plane dimensions (i.e. 63 ×80 ft). There are three bays in the x-direction with 21 feet center-to-center span length and four bays in the y-direction of 20 feet center-to-center span length (see Figure 3-2), but differ in their lateral force resisting systems (see Figure 3-1) as described below:

- Building 1 uses the moment resisting frame (MRF) which consists of beams and columns that are rigidly connected. Building 1 has twenty columns fourteen in exterior walls and six in internal walls (see Figure 3-2 a).
- Building 2 uses a combination of shear walls and MRF where the shear walls are located only on the exterior of the floor plan. Building 2 has twelve columns six in the exterior walls and six in the internal walls (see Figure 3-2 b). The shear walls in $x$-direction 14 feet long and 1.33 feet thick and shear walls in $y$-direction 20 feet long with same thickness (see Figure 3-5).

- Building 3 mainly uses the shear walls to resist lateral loads which has the both interior and exterior shear walls. Exterior shear walls have the same arrangement as building 2, but all the internal columns are replaced by the shear walls. Thus, building 3 has six columns in the exterior walls (see Figure 3-2 c).
Figure 3-1. Elevation view of building 2 and 3 (a) x-direction and (b) y-direction
(a) Building 1

(b) Building 2
Figure 3-2. Plan views of the three building models (a) building 1, (b) building 2 and (c) building 3

These three building configurations were compared because researches are still researching on the effective shear wall configurations and the amount of shear wall should be provided to obtain an acceptable building resistance against seismic attacks. For example, Murty et al. (2006) mentioned that providing symmetrical shear walls in exterior walls (same as building 2) increase buildings’ resistance against earthquake attacks.

Generally adding shear walls will increase stiffness to structures. Most high-rise buildings have shear walls for the increased stiffness resulting in less drift under lateral loads. All three models are designed using STAAD Pro (2007) considering both wind and seismic loads, as well as, gravity loads. These structures are assumed to be located in Los Angeles, United States which is a high-seismic activity zone.
3.3 Analysis and Design Process in STAAD Pro

The selected building models were analyzed and designed using the STAAD Pro (2007) software which was developed by Bentley Company. The buildings were analyzed following the steps below:

- Enter geometries of the buildings and construct the buildings’ frame using beams and columns for MRF and surface for shear walls. Finite meshing (i.e. shear walls divided into small pieces) was used in modelling surfaces.
- Enter members’ cross-sectional dimensions and material types.
- Apply fixed supports to the base of columns.
- Apply gravity loads including dead and live loads and lateral loads including seismic and wind loads to the buildings’ frame.
- Define load combinations for all load cases.
- Since lateral loads are considered, rigid diaphragms were used to constrain all nodes of the same floor (i.e. nodes for the same floor displace with the same value).
- Finally, analyze the buildings to obtain the maximum positive and negative moments and the maximum shear forces of each loading combination and create internal loading envelops based on the controlling combination cases.

After finishing analysis, RC concrete design process started. STAAD Pro (2007) follows the ACI 318-05 to design RC members including beams, columns, and shear walls. Building beams were designed based on the maximum positive and negative moments for the longitudinal reinforcement and the maximum shear force for the stirrups. Building columns were designed based on the axial force and moments in both directions. Shear
walls were designed based on the meshing moments and axial forces. Following steps were used in STAAD Pro (2007) in RC design:

- Select concrete design code which is ACI 318-05 in this research.
- Define design parameters as shown in Table 3-1

<table>
<thead>
<tr>
<th>Design Parameters</th>
<th>Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clear cover for beams and columns</td>
<td>0.125 ft</td>
</tr>
<tr>
<td>Shear walls clear cover</td>
<td>0.0625 ft</td>
</tr>
<tr>
<td>Concrete compressive strength, $f_c'$</td>
<td>4 ksi</td>
</tr>
<tr>
<td>Steel yield strength, $f_y$</td>
<td>60 ksi</td>
</tr>
<tr>
<td>Shear wall reinforcement layers</td>
<td>Two layers</td>
</tr>
</tbody>
</table>

- Design building members, then check the results. If cross-section is not adequate for the required reinforcement, increase cross-section dimensions. Software designs reinforcement of all beams, columns, and shear walls as listed in Section 3.5.

3.4 Load Calculation

Various loads acting on the structure were calculated based on the location of the building, building type and building importance. In this research, the loads acting on the buildings include dead load, live load, wind load, and seismic load. Load calculation details are explained next.

3.4.1 Gravity Loads

Gravity loads consist of dead loads and live loads. The dead loads were calculated based on the members’ cross-sectional dimensions including beam, column, shear walls,
and slab multiplied by the density of the materials and the slab thickness is 7 inch (0.583 ft). The live load is 60 psf for residential buildings. Slab weight and live load were applied to the building frame separately using the floor (area) loads in STAAD Pro (2007).

3.4.2 Wind Load

Wind loads applied to the buildings as a horizontal lateral force and it was calculated according to the ASCE 7-02. The following parameters were determined from ASCE 7-02 and entered to the STAAD Pro (2007):

1. Determine basic wind speed for Los Angeles, CA which is 85 mph according to ASCE 7-02 Figure 6-1.
2. Determine importance factor for the residential buildings (category II) which is 1 according to ASCE 7-02 Table 6-1.
3. Determine exposure category which is category B for urban and suburban regions. These parameters were entered into STAAD Pro to determine wind forces for the each story. These loads were applied in x, y, -x, and -y directions to the buildings separately and wind load for the each direction was combined with the other loads.

3.4.3 Seismic Load - Equivalent Lateral Force (ELF)

Designing structural components (i.e. dimensions and reinforcement placement) requires the knowledge of loading effects acting on the structure so that a reasonable response can be estimated under the applied load. Due to the randomness of earthquakes and many uncertainties associated with the earthquake location, intense, duration, it is very difficult to determine the actual seismic loads, therefore, researchers have been developing different analysis method to predict seismic effects and structural response. There are two main types of analysis methods, namely static analysis and dynamic analysis. ELF is an
inertial force due to seismic and applied to the structural model as a static force. Assumptions are made in determining the equivalent lateral force include: building is fixed, acceleration over all building points are equal, building supports are fixed, and calculated base shear is not accurate (Vijayendra). Design factors such as geology of the area, building type or purpose, regularity of the structure, fundamental period of the structure and soil type need to be considered. ELF is more accurate for regular buildings with uniform mass and stiffness than irregular buildings whose mass is not located at the center of the building and floor to floor sizes vary (Di Julio Jr 2001). Both horizontal and vertical irregularity affect on the ELF accuracy because empirical formulas were made based on regular buildings. In addition, irregular buildings violate the assumptions of ELF method. In this research, ELF method was adopted that were realized by the STAAD Pro (2007) software following the International Code Council (2006). Parameters input to STAAD Pro (2007) was predetermined as described below.

Parameters required by ELF include mapped acceleration, site class, response modification factor and importance factor. Site-related parameters were determined according to zip code 92887 in Los Angeles, which has longitudinal and latitudinal values of -117.7294, 33.8845, respectively using United States Geological Survey (USGS) website. Response modification factors was determined according to ASCE/SEI 7-05 (Table 12.2-1) and importance factor was determined using ASCE/SEI 7-05 (Table 11.5-1) which depends on occupancy category. Occupancy category is II for residential buildings according ASCE/SEI 7-05 (Table 1.1). The parameters summary are shown in the Table 3-2.
Table 3-2 Equivalent lateral force parameters

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mapped acceleration, $S_s$</td>
<td>2.12 g</td>
</tr>
<tr>
<td>Mapped acceleration, $S_1$</td>
<td>0.806 g</td>
</tr>
<tr>
<td>Importance factor, $I$</td>
<td>1</td>
</tr>
<tr>
<td>Response modification factor, $R$</td>
<td>5</td>
</tr>
<tr>
<td>Site class</td>
<td>D</td>
</tr>
</tbody>
</table>

ELF method can be performed according to ASCE/SEI 7-05 using following procedure which STAAD Pro follows in calculating static lateral forces along the building height after entering ELF parameters.

1. Calculate building fundamental period using Eq. 3-1 for MRF RC buildings and Eq. 3-2 for shear wall buildings. This approach is approximate method to calculate fundamental period but it is allowed by ASCE 7.

$$T_a = C_T h_n^x$$  \hspace{1cm} \text{Eq. 3-1}$$

where,

$T_a$ = the approximate fundamental period

$h_n$ = the building height in feet and it is calculated above building base to the highest point.
$C_r$ and $x = \text{the parameters used in calculating} \ T_a$ that can be calculate according to ASCE/SEI 7-05 (Table 12.8-2)

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n$$  \hspace{1cm} \text{Eq. 3-2}

$$C_w = \frac{100 \sum_{i=1}^{x} \left( \frac{h_i}{h_f} \right)^2 A_i}{A_B} \left[ \frac{A_i}{1 + 0.83 \left( \frac{h_i}{D} \right)^2} \right]$$  \hspace{1cm} \text{Eq. 3-3}

where,

$A_B = \text{the building base area in square feet}$

$A_i = \text{the web area of shear wall “i” in square feet}$

$D_i = \text{the shear wall “i” length in feet}$

$h_f = \text{the shear wall “i” height in feet}$

$x = \text{the number of shear walls in the direction which is considered to calculate lateral force.}$

2. Calculate building base shear ($V$) using Eq. 3-4

$$V = C_s W$$  \hspace{1cm} \text{Eq. 3-4}

where,

$W = \text{the effective building weight}$

$C_s = \text{the coefficient of seismic response that can be calculated using Eq. 3-5 and maximum value of} \ C_s \text{ can be calculated using Eq. 3-6 and Eq. 3-7.}$
\[ C_s = \frac{S_{DS}}{R} \frac{I}{I} \quad \text{Eq. 3-5} \]

\[ C_s = \frac{S_{D1}}{T_a} \frac{R}{R} \quad \text{if } T_a \leq T_L \quad \text{Eq. 3-6} \]

\[ C_s = \frac{S_{D1} T_L}{T_a^2} \frac{R}{R} \quad \text{if } T_a > T_L \quad \text{Eq. 3-7} \]

Value of \( C_s \) shall be equal or more than 0.01 and for buildings which have \( S_f=0.6g \), \( C_s \) shall be more than

\[ C_s = \frac{0.5 S_f}{R} \frac{I}{I} \quad \text{Eq. 3-8} \]

where,

\( S_y, S_f, I, & R \) = ELF parameters defined above.

\( S_{DS} \) = Design earthquake spectral response acceleration parameter at short period.

\( S_{D1} \) = Design earthquake spectral response acceleration parameter at 1 sec period.

\( S_{DS} & S_{D1} \) can be calculated as follows :

\[ S_{DS} = \frac{2}{3} S_{MS} \quad \text{Eq. 3-9} \]

\[ S_{D1} = \frac{2}{3} S_{M1} \quad \text{Eq. 3-10} \]

\[ S_{MS} = F_a S_s \quad \text{Eq. 3-11} \]
\[ S_{M1} = F_v S_1 \]  
Eq. 3-12

Where,

\[ F_a = \text{Site coefficient for } S_a = 2.16g \text{  and site class D is 1} \]
\[ F_v = \text{Site coefficient for } S_v = 0.806g \text{  and site class D is 1.5} \]

3. Calculate vertical distribution of the static forces \((F_x)\) using Eq. 3-13

\[ F_x = C_{vx} V \]  
Eq. 3-13

\[ C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \]  
Eq. 3-14

where,

\( w_i & w_x \) = the effective weight of part of the building located in the story \( i \) or \( x \)

\( h_i & h_x \) = the height of the building above foundation to the story \( i \) or \( x \)

\( k \) = the exponent depends on the fundamental period of building and it can be calculated as follows:

For \( T_a \leq 0.5 \text{ sec} \quad k = 1 \)

For \( T_a \geq 2.5 \text{ sec} \quad k = 2 \)

For \( 0.5 \text{ sec} \leq T_a \leq 2.5 \text{ sec} \quad k \) between 1 and 2 using linear interpolation

4. Calculate horizontal distribution static force \((V_x)\) using Eq. 3-15. \( V_x \) is divided over the vertical element for the story which is considered to calculate \( V_x \) depending on the element stiffness and condition of constrain (i.e. Diaphragm).
\[ V_x = \sum F_i \]  
Eq. 3-15

where,

\[ F_i = \text{the portion of the base shear for story } i \]

ELF was selected to use in this study because buildings met all conditions available in ASCE 7-10 Table 12.6-1. The equivalent seismic forces were then statically applied to the building joints as a horizontal forces in four directions (i.e. \( x, y, -x, \text{ and } -y \) directions) and they were separately combined with the gravity loads. Figure 3-3 shows an example of the seismic force applied laterally to the building.

Figure 3-3 Seismic loads on building 1 in \( x \)-direction
3.4.4 Load Combination

Gravity forces, wind and seismic forces were combined according to the ACI 318-11 load combinations (see Eq. 3-16 to Eq. 3-21). Seismic and wind loads applied in four directions (i.e. x, -x, y, and -y directions) and each of them were combined with the other loads separately resulting in a total of eighteen load combinations.

\[ W_u = 1.4DL \] Eq. 3-16

\[ W_u = 1.2DL + 1.6LL \] Eq. 3-17

\[ W_u = 1.2DL + 1.0WL + 1.0LL \] Eq. 3-18

\[ W_u = 1.2DL + 1.0SL + 1.0LL \] Eq. 3-19

\[ W_u = 0.9DL + 1.0WL \] Eq. 3-20

\[ W_u = 0.9DL + 1.0SL \] Eq. 3-21

where,

\[ W_u = \] factored design load

\[ DL = \] dead load

\[ LL = \] live load

\[ WL = \] wind load
$SL = \text{earthquake (seismic) load}$

### 3.5 Design Results

Cross-sectional dimensions of all RC beams and columns in Building 1 are 30” x 20” and 28” x 28” respectively. Based on these cross-sections’ dimensions loads were calculated and amount of reinforcement were determined. Maximum reinforcement of each story was used for design of the same type of member (i.e. each story has its own beam, column, and shear wall details) Cross-sectional details of a typical beam and column are shown in Figure 3-4 and the amount of reinforcement in the beams and columns of all building stories are listed in the Table 3-3. All transvers reinforcement (ties and stirrups) have two legs.

![Beam cross-section](image)

(a). Beam cross-section
Building 2 has the same beams’ and columns’ cross-sectional dimensions and the same amount of reinforcement as those in building 1 at the same story. Shear walls are 14 feet wide in the $x$-direction and are 20 feet wide in the $y$-direction as shown in Figure 3-5. Thickness for all the shear walls at the first floor is 1.5 feet and for all other floors are 1.33 feet. The amount of reinforcement of all components are listed in the Table 3-5.

**Figure 3-5 Shear wall cross-section details of typical story**
Compare to building 2, building 3 has the same arrangement of exterior shear walls and all the internal columns are replaced by shear walls as well, making it a shear wall frame model. Shear walls have the same dimensions as those in building 2 with the same reinforcement, but beams’ and columns’ dimensions are different. Beams’ cross-sectional dimension is 28” x 18” and the reinforcement details are summarized in Table 3-3. Columns cross-sectional dimension is 26” x 26” and the reinforcement details are shown in Table 3-4. Beams’ and columns’ cross-sectional details of typical story are shown in Figure 3-6.

![Beam cross-section](image)  
(a). Beam cross-section

![Column cross-section](image)  
(b). Column cross-section

Figure 3-6 Building 3 typical beam and column cross-section details (a). beam cross-section (b). column cross-section.
### Table 3-3 Beams reinforcement details for three buildings

<table>
<thead>
<tr>
<th>Story no.</th>
<th>Building 1 and building 2</th>
<th>Building 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bottom Reinforcement</td>
<td>Top Reinforcement</td>
</tr>
<tr>
<td>1</td>
<td>7 # 9</td>
<td>4 # 14</td>
</tr>
<tr>
<td>2</td>
<td>4 # 14</td>
<td>5 # 14</td>
</tr>
<tr>
<td>3</td>
<td>4 # 14</td>
<td>5 # 14</td>
</tr>
<tr>
<td>4</td>
<td>5 # 11</td>
<td>5 # 14</td>
</tr>
<tr>
<td>5</td>
<td>6 # 10</td>
<td>4 # 14</td>
</tr>
<tr>
<td>6</td>
<td>3 # 14</td>
<td>6 # 10</td>
</tr>
<tr>
<td>7</td>
<td>6 # 9</td>
<td>6 # 10</td>
</tr>
<tr>
<td>8</td>
<td>7 # 8</td>
<td>7 # 9</td>
</tr>
<tr>
<td>9</td>
<td>5 # 9</td>
<td>6 # 9</td>
</tr>
<tr>
<td>10</td>
<td>8 # 6</td>
<td>5 # 9</td>
</tr>
<tr>
<td>11</td>
<td>3 # 8</td>
<td>4 # 9</td>
</tr>
<tr>
<td>12</td>
<td>5 # 6</td>
<td>7 # 5</td>
</tr>
</tbody>
</table>
Table 3-4 Columns detail for three buildings

<table>
<thead>
<tr>
<th>Story no.</th>
<th>Building 1 and building 2</th>
<th>Building 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Axial Reinforcement</td>
<td>Column ties</td>
</tr>
<tr>
<td>1</td>
<td>4 # 14 + 16 # 10</td>
<td># 4 @14” c/c</td>
</tr>
<tr>
<td>2</td>
<td>4 # 11 + 8 # 14</td>
<td># 4 @14” c/c</td>
</tr>
<tr>
<td>3</td>
<td>4 # 10 + 8 # 14</td>
<td># 4 @12” c/c</td>
</tr>
<tr>
<td>4</td>
<td>4 # 14 + 8 # 11</td>
<td># 4 @14” c/c</td>
</tr>
<tr>
<td>5</td>
<td>4 # 14 + 8 # 11</td>
<td># 4 @10” c/c</td>
</tr>
<tr>
<td>6</td>
<td>16 # 10</td>
<td># 4 @10” c/c</td>
</tr>
<tr>
<td>7</td>
<td>8 # 14</td>
<td># 4 @10” c/c</td>
</tr>
<tr>
<td>8</td>
<td>4 # 11 + 8 # 10</td>
<td># 4 @10” c/c</td>
</tr>
<tr>
<td>9</td>
<td>4 # 11 + 8 # 19</td>
<td># 4 @10” c/c</td>
</tr>
<tr>
<td>10</td>
<td>12 # 9</td>
<td># 4 @10” c/c</td>
</tr>
<tr>
<td>11</td>
<td>12 # 8</td>
<td># 4 @10” c/c</td>
</tr>
<tr>
<td>12</td>
<td>8 # 9</td>
<td># 4 @18” c/c</td>
</tr>
</tbody>
</table>
### Table 3-5 Model 1 moment resisting frame reinforcement details for all columns

<table>
<thead>
<tr>
<th>Story no.</th>
<th>Length (feet)</th>
<th>Horizontal reinforcement</th>
<th>Vertical reinforcement</th>
<th>Edges of shear wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td># 5 @ 13 in c/c</td>
<td># 10 @ 18 in c/c</td>
<td>33 # 11</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td># 6 @ 16 in c/c</td>
<td># 10 @ 18 in c/c</td>
<td>22 # 14</td>
</tr>
<tr>
<td>2</td>
<td>20</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>28 # 11</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>18 # 14</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>16 # 14</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>14 # 14</td>
</tr>
<tr>
<td>4</td>
<td>20</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>18 # 11</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>20 # 10</td>
</tr>
<tr>
<td>5</td>
<td>20</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>14 # 11</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>20 # 9</td>
</tr>
<tr>
<td>6</td>
<td>20</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>13 # 10</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>17 # 8</td>
</tr>
<tr>
<td>7</td>
<td>20</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>15 # 8</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>10 # 9</td>
</tr>
<tr>
<td>8</td>
<td>20</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>10 # 9</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>7 # 9</td>
</tr>
<tr>
<td>9</td>
<td>20</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>10 # 7</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>8 # 6</td>
</tr>
<tr>
<td>10</td>
<td>20</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>3 # 8</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>2 # 6</td>
</tr>
</tbody>
</table>
Table 3-5 Continued

<table>
<thead>
<tr>
<th>Story no.</th>
<th>Length (feet)</th>
<th>Horizontal reinforcement</th>
<th>Vertical reinforcement</th>
<th>Edges of shear wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>20</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>2 # 6</td>
</tr>
<tr>
<td>14</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td></td>
<td>2 # 8</td>
</tr>
<tr>
<td>12</td>
<td>20</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td>2 # 6</td>
</tr>
<tr>
<td>14</td>
<td># 4 @ 10 in c/c</td>
<td># 9 @ 16 in c/c</td>
<td></td>
<td>2 # 8</td>
</tr>
</tbody>
</table>

3.6 Conclusion

This chapter starts with a description of the three high-rise RC buildings that were used in this research for seismic performance assessment. The buildings, located in Los Angeles, CA, have the same plan dimensions and height with different in lateral load resisting systems. RC building members including beam, column, and shear walls were designed according to the ACI 318 code and applied loads were calculated based on ASCE 7. Then, buildings designed using the STAAD Pro (2007). ELF was used to calculate seismic forces because it is widely used in design and buildings met all ELF conditions. Finally, cross-sectional details of the three structural (beam, column, and shear wall) of typical story were drawn and results for all other buildings were listed in tables.
4 Structure Modeling using SeismoStruct

4.1 Introduction

SeismoStruct was used to model and perform seismic analysis of the three Reinforced Concrete (RC) buildings. This chapter discusses how SeismoStruct was used to model the RC buildings and perform the Incremental Dynamic Analysis (IDA). A flow chart was created showing the main steps with detailed explanations to perform IDA. To obtain an accurate model representing complex buildings, nonlinear steel and concrete materials were used in this research. Also, geometric nonlinearity (i.e. the P-delta effect) was considered because both lateral and gravity loads are applied to the models. Fiber method was used to define the members’ cross-sections such as beams, columns, and shear walls. To implement the material distribution, the force-based formulation was used. Finally, the reasons of using rigid diaphragm for slab modeling is presented.

4.2 SeismoStruct Overview

SeismoStruct, one of the Seismosoft’s collection available online, is a finite element software which is capable of determining large displacement responses for both two and three dimensional models subjected to dynamic and static loadings. SeismoStruct considers both geometric nonlinearity and material inelasticity when analyzing buildings. Four structural materials’ models are available in the SeismoStruct which are concrete, steel, fiber-reinforced plastic and shape memory alloy. In addition, it has a 3D element library with different cross-sectional configurations for concrete, steel and composite structural members. To obtain a realistic model of a prototype building, SeismoSoft uses spread inelasticity distribution along cross-sectional and members’ length. Loadings applicable in SeismoStruct include static forces and/or displacements and dynamic accelerations.
Different analysis approaches for structural dynamic analysis and seismic response prediction are implemented in SeismoStruct such as:

1. Modal (Eigenvalue) analysis
2. Static analysis (non-variable load)
3. Static pushover analysis
4. Static adaptive pushover analysis
5. Static time history analysis
6. Dynamic time history analysis
7. Incremental Dynamic Analysis (IDA)
8. Response Spectrum Analysis

In this research, IDA was conducted in SeismoStruct for probabilistic seismic analysis of the RC buildings, which consisted of three main steps as schematically shown in Figure 4-1. First step is **pre-processing** in which all building model data were entered such as building material, member dimensions, applying loadings, and so on. Second step is **processing** in which the software performs numerical calculations, shows deformed shapes and plot displacement curves for each loading step in real time. During the processing step, users can pause or stop the analysis and check the results then re-run the analysis at the same time when the analysis is paused. **Post-processing** is the last step which displays analysis results with user-defined plots. The analysis results can be exported to Microsoft Excel for further analysis if needed. Moreover, users can create an AVI video of the building deforming shape in the post-processing step. The software is completely graphical and it does not need any command script.
Figure 4-1 IDA implementation structure in SeismoStruct
4.3 Geometric Nonlinearity

Geometric nonlinearity, also known as the P-delta effect, will severely influence building responses when subjected to lateral loads. Existing lateral loads due to seismic and/or wind loads and axial forces due to gravity loads increase the possibility of buckling in the building members. To predict the P-delta effects on the structural seismic analysis, geometric nonlinear shall be modeled.

P-delta effect is generally considered as additional overturning moments. This second order effect (compared to moment due to gravity loads considered as first order response) will increase displacement responses and extend the building natural periods (Davidson et al. 1992). Due to the complexity in modeling the P-delta effects, most designers neglect it during analysis leading to uneconomic design. For example, RC buildings analyzed in the linear elastic analysis way neglecting the P-delta effects will lead to the design of more lateral supports such as shear walls or braces to resist buckling (Dinar et al. 2013). Since this research adopts high-rise buildings (12 story) and both gravity and lateral loads are applied to the building models to accurate seismic response assessment, p-delta effect were considered.

P-delta effects are affected by the applied loads, building features and other parameters such as building height, stiffness and asymmetry degree. It shall be considered especially for those buildings carrying heavy gravity loads such as high-rise RC buildings. Currently, software are developed to analyze and design buildings with the option to include P-delta effects that are generally being categorized into two types:

1. Simply-Supported Beam P-Delta
In this type, compression axial force is applied to the beam and uniform distributed loads and/or point loads are applied to the length of beam. Both loads cause to deform the beam making beam to buckle easily. Both ends of the beam are restrained (i.e. deformation at supports is zero) (see Figure 4-2).

Figure 4-2 Simply-Supported P-Delta Effect

2. Cantilevered Column P-Delta

Compression axial force is applied to the ends of the column and uniform distributed and/or point loads are applied to the length of column which increase the deformation of the column. One support is restrained against deformation and the other support is free which gives maximum deformation (see Figure 4-3).
In this research, cantilevered column P-delta effect was considered on the RC columns (and shear walls) when gravity loads introduce compression axial force and lateral loads perpendicularly applied to the columns and shear walls. Fixed support was adopted for the foundations that did not have any deformations. Top of the building is free to move both translationally and rotationally which has the maximum deformation.

SeismoStruct provides an option to take the P-delta effects into account resulting from large displacements or rotations and big deformations relative to the frame element’s chord. P-delta effect was adopted in this research through the employment of a total co-rotational approach available in the SeismoStruct. This approach, developed and implemented by Correia and Virtuoso (2006), depends on the description of kinematic transformations associated with large displacements and the three dimensional rotation of
the building frame members which exactly defines the element’s independent deformations and forces.

4.4 Material Inelasticity

Distributed inelasticity elements is widely used in the earthquake engineering researches. Advantage of distributed inelasticity element is that it does not need to calibrate the empirical response parameters against the response of a frame element which is actual or ideal and subjected to an idealized loading case. In this study, a fiber method was adopted to model the cross-sections of the building members, during which a cross-section was divided into 150 tiny fibers. Each fiber was associated with a uniaxial nonlinear material stress-strain relationship. By integrating the nonlinear uniaxial stress-strain response of single fibers over the cross-section, the sectional stress-strain state was developed for both beams and columns (Seismosoft, 2014). Figure 4-4 illustrates a discretization of a typical RC element cross-section:
Two finite element formulations are used to implement the inelasticity distribution of structural elements which are displacement-based (DB) formulation and forced-based (FB) formulation. DB formulation is classical while FB formulation was developed more recently.

In this research, FB formulation was selected to implement the inelasticity distribution along the structural elements. FB formulation imposes a linear moment variation and it does not need any restrains along the building members. Both DB and FB formulations have the same results in the linear elastic range. However in the inelasticity range FB formulation can produce real deformed shape while DB formulation cannot. The FB formulation does not depend on the stress and strain states of individual fiber and the values of calculated sectional curvatures. This FB approach is always exact as it does not rely on the assumption of the sectional constitutive behavior. This approach has one approximation which is the discrete number of the controlling sections throughout the members to perform the numerical integration. In fact, to prevent under integration, at least three Gauss-Lobatto integration sections are required which is used widely to calculate the response of force based elements (Scott 2011). However in many cases this number is not enough to simulate the spread of inelasticity. Therefore, it is better to use a minimum of four integration points and the typical numbers of integration section are five to seven among which five sections was used in this research. This property makes each structural element to be modeled with a single FE element that allows a one to one correspondence between building members including beams, columns, and shear walls. It means that
meshing is not required within each element because FB formulation is always exact (Seismosoft 2014). Figure 4-5 shows a typical element model with six Gauss-Lobatto integration sections.

![Figure 4-5 Gauss-Lobatto integration sections (Seismosoft, 2014)]

4.5 **Material Types**

SeismoStruct has eleven material models ranging from concrete, steel, fibered-reinforced plastic to shape memory alloy. In this research, the nonlinear steel and nonlinear concrete models were used to model the RC buildings and details of both types are described below:

4.5.1 **Nonlinear Concrete Model (con_ma)**

The concrete model used in this research is a uniaxial nonlinear confinement model. This concrete model was programmed by Madas (1993) using both constitutive relationship and cyclic rules proposed by Mandar et al (1988) and Martinez-Rueda and Elnashai (1997), respectively. In addition, the effect of transverse reinforcement is
incorporated by a method proposed by Mandar et al (1988) based on the assumption that
the model has constant confining pressure throughout the whole stress and strain range
(Seismosoft 2014). Five model parameters were determined as listed in Table 4-1 to define
the material mechanical properties and the stress-strain relationship for this concrete model
is shown in Figure 4-6.

Table 4-1 Concrete parameters and their typical value (Seismosoft 2014).

<table>
<thead>
<tr>
<th>Material parameters</th>
<th>Typical value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength</td>
<td>15000 – 45000 kPa</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>2000 – 3000 kPa</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>18000 – 30000 MPa</td>
</tr>
<tr>
<td>Strain at peak stress</td>
<td>0.002 – 0.0022 (m/m)</td>
</tr>
<tr>
<td>Specific weight</td>
<td>24 kN/m3</td>
</tr>
</tbody>
</table>
4.5.2 Monti-Nuti Steel Model (stl_mn)

Monti-Nuti steel model is a uniaxial steel model that has the ability to describe post-elastic buckling of steel bars in RC buildings subjected to compression loads. Monti et al (1996) was the first one program this material model based on the work of Menegotto and Pinto (1973) that defines the stress-strain relationship with the buckling rules, the isotropic hardening rules and the additional memory rule. In the beginning, this steel model was developed for ribbed steel bars then calibrated to use for smooth steel bars. For complex RC building, it is recommended to use this type of steel model (Seismosoft 2014). But shall be confined for the RC elements which are expected to have reinforcement buckling such as columns when subjected to severe cyclic loads. Table 4-2 shows the steel model
parameters and their typical values and Figure 4-7 shows the stress-strain relationship of Monti-Nuti steel model.

<table>
<thead>
<tr>
<th>Material parameters</th>
<th>Typical value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity</td>
<td>(2.00E+08 - 2.10E+08) kPa</td>
</tr>
<tr>
<td>Yield strength</td>
<td>(230000 – 650000) kPa</td>
</tr>
<tr>
<td>Strain hardening parameter</td>
<td>0.005-0.015</td>
</tr>
<tr>
<td>Kinematic/isotropic weighing coefficient</td>
<td>0.9</td>
</tr>
<tr>
<td>Fracture strain</td>
<td>0.1</td>
</tr>
<tr>
<td>Transition curve initial shape parameter</td>
<td>20</td>
</tr>
<tr>
<td>Transition curve calibrating coefficient, A1</td>
<td>18.5</td>
</tr>
<tr>
<td>Transition curve calibrating coefficient, A2</td>
<td>0.05 – 0.15</td>
</tr>
<tr>
<td>Specific weight</td>
<td>78 kN/m3</td>
</tr>
</tbody>
</table>

Table 4-2 Steel parameters and their typical value (Seismosoft 2014).
4.6 Rigid Diaphragm and Slab Weight

4.6.1 Rigid Diaphragm

There are three types of elements in a three dimensional building model which are vertical elements, horizontal elements and foundations. Diaphragms belong to horizontal elements which transfer lateral loads from the floor system to the vertical elements such as columns or shear walls, from where these loads are further transferred to foundations.
Diaphragms also tying vertical elements together to resist lateral forces. Therefore, diaphragm is an essential element of the structure that needs to be considered during dynamic analysis.

In this research, rigid diaphragm for three buildings was assumed (i.e. slabs of each story has the same displacement). Each vertical element such as a shear wall and a column deforms separately with different displacement value if rigid diaphragm is not enforced (Moehle et al 2010). For example, shear walls have a certain displacement magnitude in its own direction, at the same time beams and columns deform to a different direction or magnitude. To obtain similar displacement within different vertical elements including columns and shear walls, rigid diaphragm was adopted in modeling the slab in this research to make the building elements displace in the same direction and same magnitude at the same floor. Rigid diaphragms were applied to building models at all floors through the use of master and slave nodes (i.e. one joint for each floor was assumed to be the master node and the other nodes in the same floor are slave nodes). These slave nodes were constrained by the master node so that slave nodes have the same displacement of master node.

4.6.2 Slab Weight

As mention previously, rigid diaphragms were adopted to model the slabs in the three building models. According to the SeismoStruct user’s manual, this is exactly the behavior in RC buildings. Slabs weight and other gravity loads acting on the slab are directly transferred to the frame elements that become lumped mass on the joints where slabs are connected to the beams and columns. Lumped mass is a single node elements which has three translational and rotational inertias values for dynamic analysis purpose (Seismosoft 2014). The weights act on the slabs consisted of the slab weight (dead load)
and 25% of the live load for the nonlinear dynamic time history analysis. Dead and live loads are then converted into lumped masses and placed on the columns. Three different lumped masses cases are used for the building models which were corner, exterior, and interior columns as shown in Figure 4-8. The hatched area represents the area of slab weight that were applied as lumped mass to the columns. Lumped masses calculations for these three cases are summarized in Table 4-3.

**Table 4-3 Lumped mass magnitudes**

<table>
<thead>
<tr>
<th>Lumped mass</th>
<th>Magnitude (Kips.sec² / in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corner lumped mass</td>
<td>0.0291</td>
</tr>
<tr>
<td>Exterior lumped mass</td>
<td>0.0571</td>
</tr>
<tr>
<td>Interior lumped mass</td>
<td>0.1163</td>
</tr>
</tbody>
</table>
Conclusion

This chapter explains the general capabilities of the SeismoStruct software and how it was used to model the three RC buildings designed in Chapter 3. Procedure of performing the IDA in SeismoStruct was briefly explained. To reasonably model the RC buildings, both geometric nonlinearity (i.e. the P-Delta effect) and the material inelasticity were adopted, where the material inelasticity uses the fiber method to model cross-section. Force-based finite element formulation was selected to implement material inelasticity along the element length. Both nonlinear steel and nonlinear concrete models were...
employed and their properties were described including the stress-strain relationship curves. Finally, the effect of the rigid diaphragm on the building deformation response were discussed. The lumped mass calculation necessary for dynamic analysis were illustrated showing the transferring of slab weights to the columns, beams and shear walls.
5 Seismic Analysis of Building Models

5.1 Introduction

Three seismic analysis were performed on the three building models including dynamic model analysis, Incremental Dynamic Analysis (IDA) and fragility assessment. Modal analysis was performed to determine the fundamental periods of the buildings which is an essential step to select and scale ground motions input for the time history analysis in the IDA. Also, all mode shapes were checked to confirm the rational of the building models established in Chapter 4 and their numerical stability. IDA method was selected to analyze the buildings because it is the most accurate method to determine building responses and visualize these responses from elastic to inelastic then collapse. To perform IDA, a number of ground motions shall be selected and they are scaled to yield comparable IDA results for the all selected ground motions. Since the buildings are designed to resist future earthquakes, probabilistic analysis is essential to expect future damage due to earthquakes. To show the probability of damage or exceeding any limit state a fragility assessment is performed by creating the fragility curves.

5.2 Modal Analysis

Modal analysis is an elastic structural dynamic analysis. Material properties remain constant within the elastic range throughout the entire calculations. Modal analysis was performed to assure from the buildings modeling and the numerical stability. Also, modal analysis was performed to determine the fundamental period of the buildings which was used for the ground motions selection and scaling.

SeismoStruct implements both Lanczos and Jacobi algorithms and the latter with Ritz Transformation. In this research, Lanczos algorithm was selected to perform modal
analysis which is capable of calculating large eigenvalue problems and determining their eigenvectors. According to Seismosoft (2014) ten modes is generally enough to capture interested mode shapes for regular buildings. In addition, the fundamental period and eighth mode was used for Raleigh’s damping during building modelling.

5.2.1 Modal Analysis Results

The fundamental period and eighth period were determined for the three RC building models (see Table 5-1) which were used in Rayleigh’s damping and for each building three modal shapes were captured in the X-direction, Y-direction and plane rotational direction which are fundamental modals shape of the buildings. It was observed that the fundamental periods decreased from Building 1 to Building 3 when increasing the amount of shear walls resulting in increased stiffness.

Table 5-1 First eight building periods for three models.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Period (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Building 1</td>
</tr>
<tr>
<td>1</td>
<td>0.93</td>
</tr>
<tr>
<td>8</td>
<td>0.17</td>
</tr>
</tbody>
</table>

Table 5-2 illustrates the fundamental three mode shapes in the three directions (i.e. along the x- and y- axes and the plane rotational direction around the z-axis) of the three RC building models. The reasonable model shapes further demonstrate the rational of the numerical models described in Chapter 4.
Table 5-2 Model deformed shapes for first five modes of three buildings

<table>
<thead>
<tr>
<th>Modes</th>
<th>Building 1</th>
<th>Building 2</th>
<th>Building 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td><img src="image1.png" alt="Image" /></td>
<td><img src="image2.png" alt="Image" /></td>
<td><img src="image3.png" alt="Image" /></td>
</tr>
<tr>
<td>2</td>
<td><img src="image4.png" alt="Image" /></td>
<td><img src="image5.png" alt="Image" /></td>
<td><img src="image6.png" alt="Image" /></td>
</tr>
<tr>
<td>3</td>
<td><img src="image7.png" alt="Image" /></td>
<td><img src="image8.png" alt="Image" /></td>
<td><img src="image9.png" alt="Image" /></td>
</tr>
</tbody>
</table>
5.3 Incremental Dynamic Analysis (IDA)

IDA is a computational analysis method of earthquake engineering for performing a comprehensive assessment of the behavior of structures under seismic loads. Simulated building seismic responses obtained from IDA are represented by IDA curves that require a series of non-linear time history analysis with a suite of ground motions, during which the ground motions’ intensities are increased using a specified scale factor. Therefore, IDA provides the buildings’ seismic behavior for the whole range from elastic to collapse.

Three steps were used to perform IDA and develop IDA curves, namely pre-process, process, and post-process as shown in Figure 5-1.
**Pre-process**
1. Create a dynamic structural model for each RC building
2. Select a suite of ground motion records
3. Scale all ground motion records to a defined design spectrum for a location where structure is located to yield comparable IDA curves
4. Choose an incremental scale factor step

**Process**
1. Perform non-linear time history analysis for each ground motion multiplied by first step of incremental scale factor
2. Increase ground motion intensity through scale factor
3. Repeat step 2 until non-converging of numerical calculations occurred which means structure has collapsed

**Post-process**
1. Choose intensity measure (IM) and damage measure (DM)
2. Plot IDA curve by interpolating the results of IM and DM for each ground motion
3. Define different limit-states for each IDA curve
4. Plot fragility curves for all determined limit states in order to calculate probability of reaching specific limit states

**Figure 5-1 Main steps used to perform IDA**
5.3.1 Nonlinear Time History Analysis

Nonlinear time history analysis is the most accurate method used to predict seismic responses of structures subjected to ground motions. Development of computer software causes to use this method widely in design new buildings and evaluating building performances during the past decade. To perform nonlinear time history analysis, ground motions directly applied to the model and it needs a suitable ground motions. Selecting ground motions is major issue in nonlinear time history analysis (Shi, 2013).

There are two methods to obtain dynamic responses of a structural model, which are direct time integration and modal superposition. The nonlinear time history analysis presented herein belong to the direct integration method which is a second order differential equation. The equation of motion for a structural system represented by MDOF model are shown in Eq. 5-1:

\[ M\ddot{x}(t) + C\dot{x}(t) + Fs(x, \dot{x}) = -M\ddot{u}_g(t) \]  

Eq. 5-1

where:

\( M \) = the mass matrix

\( C \) = the damping matrix

\( Fs \) = the resisting force of an elastoplastic system

\( \ddot{u}_g \) = the earthquake ground acceleration

\( x(t) \) = is displacement and \( \dot{x} \) is representing first derivative of the displacement which is velocity and \( \ddot{x} \) is second derivative of displacement which is acceleration, both are varying with time change.
In this study, a direct integration method was adopted to solve the equation of motion which is a second order differential equation. It is a common method used to solve dynamic response systems and it solves equation of motion numerically using discrete time stepping starting from zero to infinity. This method generally uses constant time stepping and it is not exact procedure. It can be classified into explicit and implicit methods. Researchers showed that the implicit method is more accurate than the explicit method (Fathieh 2013).

In this study, $\alpha$-integration algorithm were selected in SeismoStruct software and it developed by Hilber et al (1977). This algorithm is based on the Newmark method (i.e. has the same finite difference expression and use the same $\gamma$ and $\beta$ parameters) by adding the parameter ($\alpha$) to introduce numerical damping and improve second order accuracy and A-stability (De Jalón and Bayo 1994). Values of the three parameters shall be chosen to obtain high accuracy, numerical damping and analytical stability. The best choice for ($\alpha$) is between $[-1/3, 0]$ (this research used $\alpha = -0.1$) and the other two parameters can therefore be determined using Eq. 5-2 and Eq. 5-3 (Hilber et al 1977) that was confirmed by other researchers.

\[
\gamma = \frac{(1-2\alpha)}{2} \quad \text{Eq. 5-2}
\]

\[
\beta = \frac{(1-\alpha)^2}{4} \quad \text{Eq. 5-3}
\]

5.3.2 **Ground Motion Selection**

A suite of ground motion records needs to be selected to perform IDA and obtain reasonable results. Deciding on the number of ground motion records to be used in
nonlinear time history analysis is a subject that researchers are debating. Sufficient number of ground motions mostly depends on the application of the structure, for example, the type of the structure response, whether distribution or mean value of the responses are needed, the accuracy of the response, the expected degree of inelastic response, and the possible estimation of maximum response or collapse response (Haselton et al 2012). Therefore, each study uses different number of ground motions. For instance, according to Vamvatsikos and Cornell (2005), for mid-rise buildings ten to twenty ground motion records are usually enough to estimate seismic performance with sufficient accuracy and Haselton et al (2012) have mentioned that at least seven ground motions should be used. In this research sixteen far field ground motions were selected and each has two horizontal components which are orthogonal to each other. The following criteria was used to choose the sixteen far field ground motions (FEMA p695).

1. Source magnitude (M): Large magnitude of the earthquake can damage the structure and produce much longer duration of strong shaking and larger amount of energy released. Ground motion which has a magnitude smaller than 6.5 can damage the non-structural elements, but whole structural collapse needs greater magnitude of earthquake for new structures. Thus, all selected ground motions have a magnitude more than 6.5.

2. Site Condition: Site conditions for the ground motion records are classified into five classes (see Table 5-3) based on soil properties. The properties are determined based on soil samples located 30 meter (100 feet) below the ground surface. If there is not enough data to determine site classes, site class D should be used according to ASCE/SEI 7-10 which is the case in this study.
### Table 5-3 Site condition classification

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Site Condition or description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Hard rock</td>
</tr>
<tr>
<td>B</td>
<td>Rock (few records available for this site)</td>
</tr>
<tr>
<td>C</td>
<td>Soft rock</td>
</tr>
<tr>
<td>D</td>
<td>Stiff soil</td>
</tr>
<tr>
<td>E</td>
<td>Soft soil</td>
</tr>
<tr>
<td>F</td>
<td>Sites susceptible to ground failure</td>
</tr>
</tbody>
</table>

3. **Source Type or Fault Type:** Ground motions were recorded with different source types such as strike-slip, reverse (thrust), normal, strike-slip and reverse, strike-slip and normal and reverse and normal. Most earthquakes have strike-slip and thrust source type. These sources are typical of shallow crustal ground motions in the west side of the United States and California.

4. **Site-Source Distance:** Site to source distance is one of the most important factor to be considered while selecting the ground motion records because by this factor ground motions are divided into two groups, which are near-field and far field records. Ground motions which have site to source distance more than 10 km is classified as far-field ground motions and for records with distance less than 10 km are near-field ground motion. When conduction IDA, near-field and far-field shall be applied separately because each has different criteria for analysis.
5. Number of Records per Event: Since the location of the earthquake source is not unknown, many instruments are distributed over the region of the earthquake in order to record it with sufficient accuracy. As a result, for one earthquake many ground motions are recorded which have almost the same properties. To obtain more accurate results in the nonlinear time history analysis and to avoid potential event-based bias in record sets no more than two records are selected per one event.

6. Strongest Ground Motion Record: Selecting specific Peak Ground Motion and Peak ground Velocity is random. For example, for the 22 ground motion records chosen by FEMA p695, PGA are more than 0.2g and VGA are more than 15 m/s. However, they were selected randomly, for new structure they represents threshold of structure damage and capture large enough sample of the strongest earthquake to permit record to record variability.

In this research, sixteen ground motions were selected from the PEER ground motion database which has thousands of ground motions with different properties such as magnitude, site-source distance, site condition, and so on as explained in the ground motion selection criteria. Also, PEER database has ground motions with vertical components, the rotated fault-normal and fault-parallel motion components. Table 5-4 summarizes the criteria which was considered to select the sixteen ground motion pairs.
Table 5-4 Criteria value or type which is considered to select ground motions

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Values or Types</th>
</tr>
</thead>
<tbody>
<tr>
<td>Magnitude</td>
<td>6.5 – 7.5</td>
</tr>
<tr>
<td>Site class</td>
<td>D</td>
</tr>
<tr>
<td>Source type</td>
<td>Strike-slip, thrust</td>
</tr>
<tr>
<td>Source distance</td>
<td>More than 10 km</td>
</tr>
<tr>
<td>PGA</td>
<td>More than 0.2g</td>
</tr>
<tr>
<td>VGA</td>
<td>More than 30 m/sec</td>
</tr>
<tr>
<td>Number of records per event</td>
<td>2</td>
</tr>
</tbody>
</table>

Selected ground motions were recorded in different places around the world. Earthquake names, years of occurrence, recording station, magnitude, site conditions, source type, site source distance, PGA and VGA are shown in Table 5-5.
<table>
<thead>
<tr>
<th>GM ID</th>
<th>Earthquake Name</th>
<th>Year</th>
<th>Recording Station Name</th>
<th>M</th>
<th>Site condition</th>
<th>Source (Fault Type)</th>
<th>Site-Source Distance (km)</th>
<th>PGA (g)</th>
<th>PGV (cm/s.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Northridge</td>
<td>1994</td>
<td>Beverly Hills - Mulhol</td>
<td>6.7</td>
<td>D</td>
<td>Thrust</td>
<td>17.2</td>
<td>0.52</td>
<td>63</td>
</tr>
<tr>
<td>B</td>
<td>Northridge</td>
<td>1994</td>
<td>Canyon Country-WLC</td>
<td>6.7</td>
<td>D</td>
<td>Thrust</td>
<td>12.4</td>
<td>0.48</td>
<td>45</td>
</tr>
<tr>
<td>C</td>
<td>Duzce, Turkey</td>
<td>1999</td>
<td>Bolu</td>
<td>7.1</td>
<td>D</td>
<td>Strike-slip</td>
<td>12.4</td>
<td>0.82</td>
<td>62</td>
</tr>
<tr>
<td>D</td>
<td>Imperial Valley</td>
<td>1979</td>
<td>Delta</td>
<td>6.5</td>
<td>D</td>
<td>Strike-slip</td>
<td>22.5</td>
<td>0.35</td>
<td>33</td>
</tr>
<tr>
<td>E</td>
<td>Imperial Valley</td>
<td>1979</td>
<td>El Centro Array #11</td>
<td>6.5</td>
<td>D</td>
<td>Strike-slip</td>
<td>13.5</td>
<td>0.38</td>
<td>42</td>
</tr>
<tr>
<td>F</td>
<td>Kobe, Japan</td>
<td>1995</td>
<td>Shin-Osaka</td>
<td>6.9</td>
<td>D</td>
<td>Strike-slip</td>
<td>28.5</td>
<td>0.24</td>
<td>38</td>
</tr>
<tr>
<td>G</td>
<td>Kocaeli, Turkey</td>
<td>1999</td>
<td>Duzce</td>
<td>7.5</td>
<td>D</td>
<td>Strike-slip</td>
<td>15.4</td>
<td>0.36</td>
<td>59</td>
</tr>
<tr>
<td>H</td>
<td>Landers</td>
<td>1992</td>
<td>Yermo Fire Station</td>
<td>7.3</td>
<td>D</td>
<td>Strike-slip</td>
<td>23.8</td>
<td>0.24</td>
<td>52</td>
</tr>
<tr>
<td>I</td>
<td>Landers</td>
<td>1992</td>
<td>Coolwater</td>
<td>7.3</td>
<td>D</td>
<td>Strike-slip</td>
<td>20</td>
<td>0.42</td>
<td>42</td>
</tr>
<tr>
<td>GM ID</td>
<td>Earthquake Name</td>
<td>Year</td>
<td>Recording Station Name</td>
<td>M</td>
<td>Site condition</td>
<td>Source (Fault Type)</td>
<td>Site-Source Distance (km)</td>
<td>PGA (g)</td>
<td>PGV (cm/s.)</td>
</tr>
<tr>
<td>-------</td>
<td>----------------</td>
<td>------</td>
<td>-------------------------</td>
<td>----</td>
<td>----------------</td>
<td>---------------------</td>
<td>--------------------------</td>
<td>---------</td>
<td>------------</td>
</tr>
<tr>
<td>J</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>Capitola</td>
<td>6.9</td>
<td>D</td>
<td>Strike-slip</td>
<td>35.5</td>
<td>0.53</td>
<td>35</td>
</tr>
<tr>
<td>K</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>Gilroy Array #3</td>
<td>6.9</td>
<td>D</td>
<td>Strike-slip</td>
<td>12.8</td>
<td>0.56</td>
<td>45</td>
</tr>
<tr>
<td>L</td>
<td>Superstition Hills</td>
<td>1987</td>
<td>El Centro Imp. Co.</td>
<td>6.5</td>
<td>D</td>
<td>Strike-slip</td>
<td>18.5</td>
<td>0.36</td>
<td>46</td>
</tr>
<tr>
<td>M</td>
<td>Superstition Hills</td>
<td>1987</td>
<td>Poe Road (temp)</td>
<td>6.5</td>
<td>D</td>
<td>Strike-slip</td>
<td>11.7</td>
<td>0.45</td>
<td>36</td>
</tr>
<tr>
<td>N</td>
<td>Cape Mendocino</td>
<td>1992</td>
<td>Rio Dell Overpass</td>
<td>7.0</td>
<td>D</td>
<td>Thrust</td>
<td>14.3</td>
<td>0.55</td>
<td>44</td>
</tr>
<tr>
<td>O</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>CHY101</td>
<td>7.6</td>
<td>D</td>
<td>Thrust</td>
<td>15.5</td>
<td>0.44</td>
<td>115</td>
</tr>
<tr>
<td>P</td>
<td>San Fernando</td>
<td>1971</td>
<td>LA - Hollywood Stor</td>
<td>6.6</td>
<td>D</td>
<td>Thrust</td>
<td>25.9</td>
<td>0.21</td>
<td>19</td>
</tr>
</tbody>
</table>
5.3.3 Ground Motion Scaling

According to the ASCE/SEI 7-10, the mean of all the sixteen ground motions’ spectral acceleration should be more than the design spectrum acceleration for the location where the buildings are located within the interval of $0.2T_i$ to $1.5T_i$, where $T_i$ is the fundamental period of the building. Therefore, all ground motions are scaled to match the requirements. Then, these ground motions are scaled to the fundamental period of the three buildings separately in order to reduce the difference between design response spectrum and ground motions’ response spectrum. Procedure for scaling ground motions to match their mean to the design response spectrum and scaling to the fundamental period is shown in Figure 5-2.

1. Define design response spectrum
2. Determine fundamental period of three buildings
3. Scale sixteen ground motions’ response spectrum to design response spectrum with $0.2T_i$ to $1.5T_i$ interval
4. Scale ground motions to fundamental period to reduce difference between design spectrum and ground motions’ spectrum
5.3.3.1 Design Response Spectrum

The target response spectrum considered in this research is the design response spectrum for Los Angeles, California with zip code 92887 which has longitudinal and latitudinal values equal to -117.7294, 33.8845, respectively and site class is D. Design response spectrum of the given location is shown in Figure 5-3 with a detailed procedure of creating this design response spectrum following the ASCE/SEI 7-10 section 11.4.5:

1. Determine the mapped parameters \((S_S, S_1)\), respectively based on the given location and the site class D, which are \(2.12g\) and \(0.806g\) respectively.

   Where,

   \(S_S\) = Mapped response spectrum acceleration parameter at short period.

   \(S_1\) = Mapped response spectrum acceleration parameter at 1 second period.

2. Adjust values determined in step 1 for the site class effect using Eq. 5-4 and Eq. 5-5:

   \[
   S_{MS} = F_a S_s \quad \text{Eq. 5-4}
   \]

   \[
   S_{M1} = F_v S_1 \quad \text{Eq. 5-5}
   \]

   Where,

   \(F_a\) = Site coefficient for \(S_S = 2.16g\) and site class D is 1

   \(F_v\) = Site coefficient for \(S_1 = 0.806g\) and site class D is 1.5

3. Calculate design earthquake spectral response acceleration parameters \((S_{DS}, S_{D1})\) using Eq. 5-6 and Eq. 5-7.
\[ S_{DS} = \frac{2}{3} S_{MS} \]  \hspace{1cm} \text{Eq. 5-6}

\[ S_{D1} = \frac{2}{3} S_{M1} \]  \hspace{1cm} \text{Eq. 5-7}

Where,

\( S_{DS} \) = Design earthquake spectral response acceleration parameter at short period.

\( S_{D1} \) = Design earthquake spectral response acceleration parameter at 1 sec period.

4. Plot the design response spectrum as shown in Figure 5-3 using the following equations:

a. When period smaller than \( T_o \), calculate design response spectral acceleration \( (S_a) \) use Eq. 5-8:

\[ S_a = S_{DS} (0.4 + 0.6 \frac{T}{T_0}) \]  \hspace{1cm} \text{Eq. 5-8}

b. When period more than or equal to \( T_o \) and smaller than or equal to \( T_s \), use \( S_a \) equal to \( S_{DS} \).

c. When period more than \( T_s \) and smaller than or equal to \( T_L \), calculate \( S_a \) use Eq. 5-9:

\[ S_a = \frac{S_{D1}}{T} \]  \hspace{1cm} \text{Eq. 5-9}

d. When period more than \( T_L \), calculate \( S_a \) use Eq. 5-10:

\[ S_a = \frac{S_{D1}T_L}{T^2} \]  \hspace{1cm} \text{Eq. 5-10}

Where,
\[ T = \text{Fundamental building period in second} \]

\[ T_0 = 0.2 \frac{S_{D1}}{S_{DS}} \]

\[ T_s = \frac{S_{D1}}{S_{DS}} \]

\[ T_L = \text{Long period transition period.} \]

**Figure 5-3 Design response spectrum for zip code 92887**

**5.3.3.2 Ground Motion Scaling Between (0.2T_1~1.5T_1)**

The SeismoMatch software was used in the ground motion scaling. SeismoMatch is one of the SeismoSoft’s collection and it has the ability to scale ground motion acceleration to match to a given target response spectrum. SeismoMatch uses the wavelets algorithm proposed by Abrahamson (1992) and Hancock et al (2006) to match a group of ground motions to the target spectrum and calculating the mean value of them at the same
time. In addition, SeismoMatch is capable of reading single accelerograms having single or multiple values per line and it can create a target spectrum response according to the Euro code 8 or user defined target spectrum. It plots time histories and response spectrums for both original and matched ground motions and it provides tables of their values which can be exported to the Excel spread sheet (Seismosoft 2014)

Matching or scaling ground motions to a target response spectrum by using SeismoMatch consists of three steps as shown in Figure 5-4. First, load all ground motions to the SeismoMatch software as a single ground motion or multiple ground motions. Second, define target spectrum in the software or load user defined target spectrum to the software. Third, enter interval which needs to match such as $0.2T_i$ to $1.5T_i$ and enter scale factor if available.
Figure 5-4 Matching ground motions to the target spectrum steps using

SeismoMatch

Response spectral acceleration of the original sixteen ground motion pairs are shown in Figure 5-5. Sixteen ground motions were then matched to the design response spectrum based on the buildings’ fundamental period using the SeismoMatch software. Each building has a different fundamental period as calculated in the modal analysis section.
(Section 5.2) resulting in different matching intervals \((0.2T_i \text{ to } 1.5T_i)\) as listed in Table 5-6.

The response spectrum of the matched ground motions for buildings 1, 2, and 3 are shown in Figure 5-6, Figure 5-7 and Figure 5-8, respectively.

### Table 5-6 First mode period and matching intervals for 3 structural models

<table>
<thead>
<tr>
<th>Structural Model</th>
<th>Fundamental period (sec)</th>
<th>Minimum (sec)</th>
<th>Maximum (sec)</th>
<th>Design Spectral Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>0.93</td>
<td>0.19</td>
<td>1.4</td>
<td>0.8582</td>
</tr>
<tr>
<td>Model 2</td>
<td>0.75</td>
<td>0.15</td>
<td>1.13</td>
<td>1.0746</td>
</tr>
<tr>
<td>Model 3</td>
<td>0.71</td>
<td>0.14</td>
<td>1.07</td>
<td>1.1451</td>
</tr>
</tbody>
</table>

**Figure 5-5** Response spectrum for the original selected ground motion pairs and the design response spectrum.
Figure 5-6 Response spectrum for building 1 all ground motions matched to the building 1 interval (0.19 sec to 1.4 sec)

Figure 5-7 Response spectrum for building 2 all ground motions matched to the building 2 interval (0.15 sec to 1.13 sec)
5.3.3.3 Ground Motion Scaling to Fundamental Period

After matching ground motions to the $0.2T_1$ to $1.5T_1$ interval, they were scaled to the acceleration corresponding to the fundamental period of the building. This scaling reduces the difference between target spectrum and selected ground motions’ mean and facilitates the comparison between ground motions’ effects on building response using the IDA. For each building, the target spectrum acceleration at the fundamental period was determined from the design spectrum shown in Figure 5-3 and all ground motions were scaled to the same acceleration at the same period by using Eq. 5-11.

$$SF = \frac{S_u(T_1)_{\text{Target}}}{S_u(T_1)_{GM}}$$  \hspace{1cm} \text{Eq. 5-11}$$

Where,
SF = Scale factor for a ground motion.

$S_a(T_1)_{\text{Target}}$ = Spectral acceleration of the target spectrum at fundamental period.

$S_a(T_1)_{\text{GM}}$ = Spectral acceleration of the ground motion at fundamental period.

For example, the fundamental period for building 1 is 0.94 seconds and the target spectral acceleration for the same period is $0.8582g$ as shown in Table 5-6. The spectrum acceleration for the Northridge ground motion component 1 is $0.8794g$, therefore, a scale factor equal to $0.8582/0.8794 = 0.98$ was determined. The scale factors of the sixteen ground motions for the three buildings are listed in Table 5-7. Spectral accelerations of the three building models after scaling to the target spectral acceleration at the fundamental period are shown in Figure 5-9, Figure 5-10 and Figure 5-11, respectively.
Table 5-7 Scale factors for sixteen ground motions with both horizontal components for three models

<table>
<thead>
<tr>
<th>GM</th>
<th>Horizontal components</th>
<th>Scale Factors (Buildings)</th>
<th>GM</th>
<th>Horizontal components</th>
<th>Scale Factors (Buildings)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>1</td>
<td>0.98</td>
<td>1.03</td>
<td>0.96</td>
<td>I</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.97</td>
<td>0.96</td>
<td>1.09</td>
<td>2</td>
</tr>
<tr>
<td>B</td>
<td>1</td>
<td>0.95</td>
<td>1.00</td>
<td>0.98</td>
<td>J</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.00</td>
<td>1.08</td>
<td>0.94</td>
<td>2</td>
</tr>
<tr>
<td>C</td>
<td>1</td>
<td>0.94</td>
<td>0.97</td>
<td>1.04</td>
<td>K</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.04</td>
<td>0.95</td>
<td>0.97</td>
<td>2</td>
</tr>
<tr>
<td>D</td>
<td>1</td>
<td>0.95</td>
<td>0.99</td>
<td>1.02</td>
<td>L</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.06</td>
<td>0.97</td>
<td>1.00</td>
<td>2</td>
</tr>
<tr>
<td>E</td>
<td>1</td>
<td>1.29</td>
<td>1.10</td>
<td>1.22</td>
<td>M</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.25</td>
<td>0.99</td>
<td>1.03</td>
<td>2</td>
</tr>
<tr>
<td>F</td>
<td>1</td>
<td>1.02</td>
<td>0.98</td>
<td>0.96</td>
<td>N</td>
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<td>0.93</td>
<td>0.99</td>
<td>1.00</td>
<td>2</td>
</tr>
<tr>
<td>G</td>
<td>1</td>
<td>1.01</td>
<td>0.97</td>
<td>1.00</td>
<td>O</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.97</td>
<td>1.03</td>
<td>0.99</td>
<td>2</td>
</tr>
<tr>
<td>H</td>
<td>1</td>
<td>0.96</td>
<td>1.06</td>
<td>0.99</td>
<td>P</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.04</td>
<td>1.13</td>
<td>1.05</td>
<td>2</td>
</tr>
</tbody>
</table>
Figure 5-9 Response spectrum for building 1 after scaling to for all ground motion

Figure 5-10 Response spectrum for building 2 after scaling to for all ground motion
Figure 5-11 Response spectrum for building 3 after scaling to for all ground motion

5.3.4 Damage Measure (DM) Selection

DM, also referred as Engineering Demand Parameters (EDP), is a measure of structural responses to the lateral loads effect such as base shear, top drift, maximum inter-story drift, and so on as described in chapter two. Selecting DM depends on the purpose of the analysis, for instance, for seismic analysis lateral deformation and story drift are most common (Kruep 2007). Performance assessment type also affects the DM selection, for example, for non-structural damage evaluation the peak roof accelerations are used and for structural damage maximum inter-story drift is a proper selection (Fathieh 2013). This research uses maximum inter-story drift ratio as the DM to compare the structural damage of the three RC buildings.
5.3.5 Intensity Measure (IM) Selection

IM is a scalar which increases monotonically with IDA scale factor. IMs of earthquake are the Richter scale or Modified Mercalli scale that can be expressed as Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) or 5% damped first-mode spectral acceleration $S_a(T_{1,5\%})$ for engineering purposes. For moderate period buildings and no near-fault ground motions, $S_a(T_{1,5\%})$ is more suitable and efficient IM than PGA (Vamvatsikos and Fragiadakis 2010). In addition, $S_a(T_{1,5\%})$ gives more consistent results than PGA (Dhakal et al 2006). Therefore $S_a(T_{1,5\%})$ was adopted in this research as the IM.

After IM was selected, the increment of the IDA scale factor shall be determined which defines the ground motions’ intensity in the nonlinear time history analysis at each intensity level. In this research 0.1 was used as the. The selection of 0.1 increment requires a long time to complete the IDA analysis that might not be needed to create the IDA curve. To decrease the number of analyses Vamvatsikos (2002) proposed a hunt-and-fill method which determines the exact number of analyses to create the IDA curve.

5.4 Fragility Curve Development

The fragility curve illustrates the probabilities of structures that reach or exceed certain limit states. Developing fragility curves requires defining uncertainties associated with ground motions and structural materials. In this study, only ground motion uncertainty was considered in creating the fragility curves and the analytical fragility curve was chosen to show the probability of damage.

5.4.1 Fragility Curve Equation Derivation

Developing the fragility curve needs vulnerability function which depends on the uncertainties and it can be derived as follows (De Leon 2010).
Cumulative Distribution Function (CDF) is calculated to show the probability of a random variable $X$ which is less than or equal to a specific value $x$:

$$ P(X \leq x) = F_X(x) = \int_{-\infty}^{x} f_X(x)dx $$  \hspace{1cm} \text{Eq.5-12}$$

By taking the derivative of the CDF function (see Eq.5-13), the probability density function (PDF) is produced which cannot provide probability of the random variable, but shows the randomness of the variable:

$$ f_X(x) = \frac{dF_X(x)}{dx} $$  \hspace{1cm} \text{Eq.5-13}$$

PDF is then expressed as the normal distribution which is the most common engineering probability distribution:

$$ f_X(x) = \frac{1}{\zeta_x \sqrt{2\pi}} \exp \left[-\frac{1}{2} \left( \frac{x - \lambda_x}{\zeta_x} \right)^2 \right] $$  \hspace{1cm} \text{Eq.5-14}$$

Where,

$\lambda_x =$ mean of variable $X$.

$\zeta_x =$ standard deviation of variable $X$

Due to difficulties in integrating PDF function (Eq.5-14), transformation of the random variable $X$ is performed into a standard normal distribution considering to have a zero mean and a standard deviation equal to one as shown below:

$$ s = \frac{X - \lambda}{\zeta} $$  \hspace{1cm} \text{Eq.5-15}$$

where,

$s =$ standard normal of variables
\( \lambda = \text{mean of variables} \)

\( \zeta = \text{standard deviation of variables} \)

\( X = \text{random variables} \)

After substituting \( \lambda = 0 \) and \( \zeta = 1 \), the standard normal distribution equation of Eq.5-14 becomes:

\[
\phi(s) = f_s(s) = \frac{1}{\sqrt{2\pi}} \exp \left[ -\frac{1}{2} (s)^2 \right] \quad \text{Eq.5-16}
\]

Since the normal distribution is symmetry, the following equation can be rewritten as:

\[
\Phi(-s) = 1 - \Phi(s) \quad \text{Eq.5-17}
\]

Where,

\( \Phi(s) = F_s(s) = \text{CDF of the standard normal distribution.} \)

Rewrite Eq.5-15 into following form

\[
X = s\zeta + \lambda \quad \text{Eq.5-18}
\]

Implement CDF definition \( (F_X(x) = P(X \leq x)) \) then substitute Eq.5-18, then perform following calculations

\[
F_X(x) = P(X \leq x) \quad \text{Eq.5-19}
\]

\[
= P(X \leq x)
\]

\[
= P(s\zeta + \lambda \leq x)
\]
\[= P(s \leq \frac{X - \lambda}{\zeta})\]

\[F_X(X) = F_S(S) = P(s \leq \frac{X - \lambda}{\zeta}) \quad \text{Eq. 5-20}\]

Since random variables in this research should not have negative sign, natural logarithmic is taken for all X values (see Eq. 5-21).

\[F_X(X) = P(Ln(X) \leq Ln(x)) \quad \text{Eq. 5-21}\]

Therefore, the CDF is expressed as:

\[F_X(x) = \phi\left(\frac{Ln(X) - \lambda}{\zeta}\right) \quad \text{Eq. 5-22}\]

### 5.4.2 Procedure for Creating Fragility Curve

In this study, the following procedure was used to create the fragility curves

1. Analyze the building models using the IDA and create the IDA curves for the sixteen ground motions in both directions. Determine the value of IM which are \(S_{a(T,5\%)}\) of the building responses from the IDA curves of the 16 ground motions and these values are used as the ground motion parameters in the fragility curve (i.e. horizontal axis).

2. To obtain the fragility curve assumption (i.e. all variables log normally distributed), natural logarithmic shall be taken \(Ln(X)\) for ground motion parameters.

3. Calculate the mean and the standard deviation for \(Ln(x)\) using Eq.5-23 and Eq.5-24:
\[
\lambda = \frac{\sum_{i=1}^{n} \ln(x_i)}{n} \quad \text{Eq.5-23}
\]

\[
\zeta = \sqrt{\frac{\sum_{i=1}^{n} (\ln(x_i) - \lambda)^2}{n-1}} \quad \text{Eq.5-24}
\]

Where,

\( \lambda \) = mean of \( \ln(x) \).

\( \zeta \) = standard deviation of \( \ln(x) \).

\( x \) = ground motion parameters could be \( S_a(T_{1,5\%}) \), base shear or any IM of IDA curve.

4. Calculate \( s \) of the lognormal data using Eq.5-25:

\[
s = \frac{\ln(x) - \lambda}{\zeta} \quad \text{Eq.5-25}
\]

5. Apply the standard normal distribution for the probability function and CDF which is denoted as \( \Phi \) using Eq.5-26

\[
P(\leq D) = \phi\left(\frac{\ln(x) - \lambda}{\zeta}\right) \quad \text{Eq.5-26}
\]

6. Plot fragility curve between probability as vertical axis and IM as horizontal axis.

5.5 Conclusion

This chapter presented the seismic analysis performed in this research of the three RC buildings. Firstly results from the modal analysis were listed that verified the numerical stability and rational of the building models setup up in Chapter 4. In addition fundamental periods were determined from the modal analysis that were used in ground motion scaling.
required by IDA. Then the detailed procedure of conducting IDA and fragility assessment was discussed. Specifically, the consideration of selecting the ground motions, the two steps of scaling ground motions and the IM and DM selections were presented. The resulting scaling factors used in the nonlinear time history analysis and the scaled ground motion spectrums required by the IDA are shown. Finally the theoretical background and fragility assessment was explained and the equations of creating fragility curves were derived based on the probability theory. The seismic analysis method presented in this chapter were applied to the three RC buildings and the results are discussed in the next chapter.
6 Seismic Analysis Results and Discussion

6.1 Introduction

This chapter presents and discusses the results of the analyses described in chapter 5 including Incremental Dynamic Analysis (IDA) and fragility assessments. Seismic responses of the three high-rise RC buildings are compared in the x and y-direction separately. Dispersion values obtained from the multi-record IDA curves are firstly calculated to examine the robustness of the buildings’ response under random earthquake inputs. Based on the IDA results, fragility curves are therefore developed to probabilistically compare buildings’ performances at the three limit states including slight damage, moderate damage, and collapse. The method to determine the limit states from the IDA curves is presented. In the end, the relationship between shear wall amount and the mean of log-normally distributed ground motion parameters are shown in extended fragility curves through a linear regression analysis to further illustrate the effect of shear wall amounts on seismic performances at the three limit states.

6.2 IDA Results and Dispersions

Following the IDA analysis, IDA curves were plotted for different Damage Measure (DM) and Intensity Measure (IM) as discussed in Chapter 5. In this research the maximum inter-story drift ratio was considered as the DM and the spectral acceleration at the fundamental period was taken as the IM. Single-record IDA curve were generated for all sixteen ground motions to locate their respective three limit states (i.e. slight damage, moderate damage, and collapse). In this chapter the multi-record IDA curves (i.e. sixteen IDA curves) of the three RC buildings are shown for both directions in Figure 6-1~6.3.
a. x-direction

b. y-direction
Figure 6-1 Multi-record IDA curve for building 1 (a) x-direction (b) y-direction

a. x-direction
b. y-direction

Figure 6-2 Multi-record IDA curve for building 2 (a) x-direction (b) y-direction

a. x-direction
Multi-record IDA curves help researchers to understand the building responses subjected to a number of ground motions considering the random feature of earthquake excitations. A smaller difference among the responses under different ground motions is an index of structural robust seismic response. Therefore, reducing record-to-record variability (dispersion) is a key point for improving robustness of the building responses that requires the IDA analysis. On the other hand, larger dispersion causes unpredictability in design (Spears 2004).

Dispersion is defined as the difference between the minimum and the maximum values of the DM or IM for a given value of IM or DM. Dispersion was calculated for the
three buildings in two directions for both IM at the given DM (4% inter-story drift) and DM at the given IM (0.4 g) (see Figure 6-1) and the results as listed in Table 6-1.

**Table 6-1 DM and IM dispersions for the three buildings in both direction.**

<table>
<thead>
<tr>
<th>Buildings</th>
<th>Direction</th>
<th>DM Dispersion, %</th>
<th>IM Dispersion, g</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building 1</td>
<td>X</td>
<td>0.52</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>0.37</td>
<td>0.51</td>
</tr>
<tr>
<td>Building 2</td>
<td>X</td>
<td>0.12</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>0.16</td>
<td>0.29</td>
</tr>
<tr>
<td>Building 3</td>
<td>X</td>
<td>0.16</td>
<td>0.44</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>0.16</td>
<td>0.45</td>
</tr>
</tbody>
</table>

DM-dispersion: Along the x-direction, building 1 has the largest DM dispersion while the other two buildings have similar values. Dispersion in building 1 is almost 4 times of the dispersion in buildings 2 and 3. Similar trend is also observed in the y direction.

IM-dispersion: In both directions, building 2 has the smallest dispersion than the other two buildings although the differences among the three buildings are smaller compared to those of the DM-dispersions.

In general, building 2 has the smallest DM and IM dispersion in both directions and buildings 2 and 3 have similar values. Considering the shear walls included in buildings 2 and 3, it may be concluded that adding shear walls generally reduces dispersion to achieve increased robustness and reduce record-to-record variability which gives predictability in design. For the two methods calculating dispersions considered herein, DM dispersion (when IM is fixed) seems to be more consistent with the expected behavior of RC buildings.
However as can be seen, the selection of the fixed IM or DM values greatly affect the dispersion results so multiple IM or DM values maybe considered in future for dispersion comparison purpose. In addition, a direct visual inspection on multi-record IDA curves may lead to similar results as the calculated DM / IM dispersion values. As IDA curves are more close to each other, it means that building model has more robust seismic response against variable earthquake inputs.

6.3 Limit States

Different damage levels (i.e. limit states) have been used by researchers such as slight damage, moderate damage, major damage, and collapse defined by HAZUS MR4. In this research, three limit states are identified in terms of the inter-story drift ratio which are slight damage, moderate damage, and complete collapse. To determine slight and moderate damage limit states, a DM-based rule was used and an IM-based rule was used to determine the collapse limit state. For the Moment Resisting Frame (MRF) buildings (building 1), the HAZUS definitions of the three limit states is as follows:

- Slight damage: Hairline cracks for shear and flexural occur in the beam and column joints or areas close to joints.

- Moderate damage: Hairline cracks are shown on most columns and beams. For non-ductile buildings larger shear cracks and some concrete spalling may occur, while for ductile buildings flexural cracks and spalling may be observed.

- Complete Collapse: building is collapsed or on the edge of collapse because of brittle failure of non-ductile buildings or building losses its stability.

For buildings with shear walls (i.e. buildings 2 and 3), HAZUS defines the three limit states as:
- Slight damage: Shear walls have diagonal hairline cracks with minor concrete spalling.

- Moderate damage: Most shear walls have diagonal cracks and some of them have larger diagonal cracks and concrete spalling showing that shear walls exceed yielding capacity.

- Complete Collapse: Building is collapsed or on the edge of collapse because of failure of most shear walls and some critical beams and columns.

HAZUS MR4 defines these limit states based on the field observations after earthquakes and it uses concrete cracks for RC buildings as measure of damage. According to these observed cracks at different damage limit state, maximum inter-story drift ratio limit was determined for three building classes (i.e. low, mid, and high-rise buildings).

As this study develops analytical fragility curves based on numerical simulation, no direct observation of crack development is available. Therefore, inter-story drift ratio of the slight and moderate damage limit states were determined according to the HAZUS (National Institute of Building Sciences, 2004) using Table 5.9a for high-code seismic design levels. This high-code seismic design levels are equivalent to seismic design category D based on ASCE 7-05. For the complete collapse, the median (50% fractile) of all IDA curves were used to determine the corresponding maximum inter-story drift ratio at which the median curves started to become a flat line (see Figure 6-4~Figure 6-6) (Karapetrou et al 2015). Inter-story drift ratios of the three limit states of the three buildings are summarized in Table 6-2.
Table 6-2 Shows inter-story ratio values for three buildings in both directions

<table>
<thead>
<tr>
<th>Building</th>
<th>Maximum Inter-Story Drift Ratio, %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Slight damage</td>
</tr>
<tr>
<td></td>
<td>X- and Y-directions</td>
</tr>
<tr>
<td>1</td>
<td>0.25</td>
</tr>
<tr>
<td>2</td>
<td>0.2</td>
</tr>
<tr>
<td>3</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Figure 6-4 Median of IDA curves and inter-story drift ratio percentage for three limit states for building 1
Figure 6-5 Median of IDA curves and inter-story drift ratio percentage for three limit states for building 2

Figure 6-6 Median of IDA curves and inter-story drift ratio percentage for three limit states for building 3
6.4 Fragility Curves

Based on the limit states identified from the IDA curves, a graphical statistical method was utilized to develop fragility curves that represent the probability of exceeding a certain limit state. In this study, fragility curves were plotted for slight damage, moderate damage and collapse limit states for the three building models in both directions against the spectral acceleration at the fundamental period with 5% damping $S_a(T_1, 5\%)$, the same used in the IDA curves. Fragility curves were expressed by assuming the log-normally distribution of the data points at $S_a(T_1, 5\%)$ on the sixteen IDA curves. The probability of exceeding a damage level (D) can be determined using Eq. 6-1 as discussed in chapter 5.

$$P(\leq D) = \phi \left( \frac{Ln(x) - \lambda}{\zeta} \right)$$  \hspace{1cm} Eq. 6-1

Where, $\phi$ = the standard normal distribution.

$x$ = the ground motion parameters, which is $S_a(T_1, 5\%)$ in this research and obtained from IDA curves, were assumed to be log-normally distributed by taking natural logarithm ($Ln(x)$).

$\lambda$ = the mean of $Ln(x)$.

$\zeta$ = the standard of $Ln(x)$.

An example is shown next to illustrate the procedure of creating fragility curve of the collapse limit state of building 1 in the $x$-direction. The detailed procedure for this example is as bellow:

- Ground motion parameters ($x$) for each ground motion at the collapse limit state after sorting from the smallest value to the biggest value and their natural logarithm ($Ln(x)$) are listed in Table 6-3.
- Mean and standard deviation for $Ln(x)$ were calculated where $\lambda = -0.32$ and $\zeta = 0.26$.

- After calculating the mean and standard deviation, the standard normal variable ($s$) is calculated using Eq.5-25 for each variable ($x$) and their values are shown in Table 6-3.

- Take standard normal distribution to calculate the probability values using the Microsoft Excel function (NORMSDIST) for each value of $s$ and the results are listed in the last column of Table 6-3.
Table 6-3 Fragility curve creation data of collapse limit state for building 1 in x-direction

<table>
<thead>
<tr>
<th>$x = S_a(T_{1,5%})$</th>
<th>$\ln(x)$</th>
<th>$s$</th>
<th>Probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.44</td>
<td>-0.82</td>
<td>-1.92</td>
<td>0.03</td>
</tr>
<tr>
<td>0.45</td>
<td>-0.80</td>
<td>-1.83</td>
<td>0.03</td>
</tr>
<tr>
<td>0.61</td>
<td>-0.49</td>
<td>-0.66</td>
<td>0.25</td>
</tr>
<tr>
<td>0.62</td>
<td>-0.48</td>
<td>-0.60</td>
<td>0.28</td>
</tr>
<tr>
<td>0.64</td>
<td>-0.45</td>
<td>-0.47</td>
<td>0.32</td>
</tr>
<tr>
<td>0.67</td>
<td>-0.40</td>
<td>-0.30</td>
<td>0.38</td>
</tr>
<tr>
<td>0.68</td>
<td>-0.39</td>
<td>-0.24</td>
<td>0.40</td>
</tr>
<tr>
<td>0.69</td>
<td>-0.37</td>
<td>-0.18</td>
<td>0.43</td>
</tr>
<tr>
<td>0.69</td>
<td>-0.37</td>
<td>-0.18</td>
<td>0.43</td>
</tr>
<tr>
<td>0.82</td>
<td>-0.20</td>
<td>0.48</td>
<td>0.68</td>
</tr>
<tr>
<td>0.84</td>
<td>-0.17</td>
<td>0.57</td>
<td>0.72</td>
</tr>
<tr>
<td>0.85</td>
<td>-0.16</td>
<td>0.62</td>
<td>0.73</td>
</tr>
<tr>
<td>0.88</td>
<td>-0.13</td>
<td>0.75</td>
<td>0.77</td>
</tr>
<tr>
<td>1.01</td>
<td>0.01</td>
<td>1.28</td>
<td>0.90</td>
</tr>
<tr>
<td>1.02</td>
<td>0.02</td>
<td>1.32</td>
<td>0.91</td>
</tr>
<tr>
<td>1.03</td>
<td>0.03</td>
<td>1.36</td>
<td>0.91</td>
</tr>
</tbody>
</table>

$\lambda = -0.32$
- Last step is to draw the relationship between the ground motion parameters ($x$) and the probability which are shown in the first and the last columns of Table 6-3. Then, the fragility curve is obtained as shown in Figure 6-7.

![Fragility curve of collapse limit state of building 1 in x-direction](image)

**Figure 6-7  Fragility curve of collapse limit state of building 1 in x-direction**

### 6.4.1 Comparing Three Limit States of the Same Building

Figure 6-8 to Figure 6-13 illustrate the fragility curves comparison of the same building at three limit states along two directions.
Figure 6-8 Fragility curves for slight damage, moderate damage and collapse damage levels of building 1 in $x$-direction

Figure 6-9 Fragility curves for slight damage, moderate damage and collapse damage levels of building 1 in $y$-direction
Figure 6-10 Fragility curves for slight damage, moderate damage and collapse
damage levels of building 2 in $x$-direction

Figure 6-11 Fragility curves for slight damage, moderate damage and collapse
damage levels of building 2 in $y$-direction
Figure 6-12 Fragility curves for slight damage, moderate damage and collapse
damage levels of building 3 in \textit{x-direction}

Figure 6-13 Fragility curves for slight damage, moderate damage and collapse
damage levels of building 3 in \textit{y-direction}
As seen from the Figure 6-8 to Figure 6-13, fragility curves corresponding to the moderate damage limit state are approximately located in the middle between the slight damage and the collapse limit states’ curves for buildings 1 and 3 along both directions. However collapse fragility curves of building 2 are close to the moderate damage limit state’s which means that building 2 is expected to collapse soon after exceeding the moderate damage level in both directions. This phenomena is also noticed in the maximum inter-story drift ratio of the collapse limit state which are listed in Table 6-2, which is close to its respective drift ratio at the moderate damage state (i.e. 0.58 and 0.59 for collapse vs. 0.5 for moderate damage). Building 2 has minimal safety margin between reaching the moderate limit state and the collapse limit state may be attributed to the irrational arrangement of shear walls at the periphery of the building plan. In addition, the maximum inter-story drift ratio defined in HAZUS for shear wall buildings might need to be reexamined as it may not provide the reasonable value corresponding to high-rise RC shear wall buildings with different shear wall configurations.

6.4.2 Comparing the Same Limit States of Three Buildings

Fragility curves of the same limit state of the three building models were plotted on the same graph to compare their seismic performances (see Figure 6-14 through Figure 6-19).
Figure 6-14 Fragility curves shows comparison of slight damage for three buildings in x-direction

Figure 6-15 Fragility curves shows comparison of slight damage for three buildings in y-direction
Based on the fragility curves of the slight damage limit state in both directions shown in Figure 6-14 and Figure 6-15, buildings 1 and 2 have similar seismic performances which means they required the same ground motion intensity to reach the slight damage limit state. While, building 3 has slight better performance at slight damage limit state which means that building 3 relatively requires higher ground motion intensity to reach the slight damage limit state. This is expected based on HAZUS MR4 definition for maximum inter-story drift ratio values defined for the slight damage limit state considering different building types (i.e. MRF and shear wall buildings). Also, the relatively vertical straight fragility curves of buildings 2 and 3 (i.e. buildings which have shear walls) illustrate the robust responses of these two building at the slight damage limit state, during which both buildings remain in the elastic range. Therefore, increasing building stiffness by adding shear walls has positive effects at the slight damage limit state, although this effect seems minimum.

Figure 6-16 Fragility curves shows comparison of moderate damage for three buildings in *x-direction*
Figure 6-17 Fragility curves shows comparison of moderate damage for three buildings in $y$-direction

According to the fragility curves developed for the moderate damage limit state in both directions of the three buildings (see Figure 6-16 and Figure 6-17), building 3 requires larger earthquake intensity to cause diagonal hair line cracks in most shear walls and concrete spalling in some shear walls (i.e. reach moderate damage limit states). However, building 1 requires the smallest earthquake intensity among the three buildings to exceed its moderate damage limit state and building 2 is exactly in between them. Therefore, adding shear walls to the buildings (i.e. increasing building stiffness), improves seismic performances effectively at the moderate damage limit state. This results is expected because the maximum inter-story drift ratio for three buildings defined by HAZUS MR4 are equal (i.e. 0.5%).
Figure 6-18 Fragility curves shows comparison of complete damage for three buildings in \textit{x-direction}.

Figure 6-19 Fragility curves shows comparison of complete damage for three buildings in \textit{y-direction}.
Based on the fragility curves for the collapse limit state in both directions of three buildings shown in Figure 6-18 and Figure 6-19, the fragility curves of buildings 1 and 2 are almost coincide which mean both buildings require the earthquake intensity to collapse the buildings, although building 2 has larger stiffness (i.e. enough amount of shear walls). While the fragility curves of building 3 at the collapse limit state are farther away from those of buildings 1 and 2 with relatively large difference which means building 3 requires a high earthquake intensity to collapse compared to the other two buildings. The fragility curves of buildings 1 and 2 at collapse limit state being so close might be attributed to that internal columns of building 2 failed before the shear wall failure at a similar drift ratio of column failure in the MRF model. Due to the rigid diaphragm assumption made in the numerical model, the same inter-story drift on the internal columns of building 2 will be experiences by the exterior shear walls, which although have larger stiffness but generally less ductility. Therefore shear wall quickly fails after the internal column failure. While building 3 has both internal and external shear walls requires a high earthquake intensity to collapse due to its high stiffness. Thus it may be concluded that increasing stiffness to the buildings might not be enough to improve seismic performance at collapse limit state without considering the configuration of the buildings and location of shear walls.

In summary, in slight damage limit state buildings 1 and 2 have similar performance and building 3 is better with small difference showing that increasing building stiffness may give more robustness to the buildings to reach slight damage limit state. The fragility curves of the three buildings at the moderate damage limit state are relatively equal displaced with building 3 being the rightest (best) and building 1 being the leftist (worst) demonstrating that adding shear wall effectively increase seismic performances at the
moderate damage limit state. As to the collapse limit state, building 3 is more stable than the other two buildings with significant difference while buildings 1 and 2 have similar seismic performance at collapse limit state. From this observation, we may conclude that although generally increase stiffness will improve seismic performance. At the collapse limit state, building plan configurations (i.e. shear wall location) may be more important than shear wall amounts that require further study.

6.4.3 Extended Fragility Curves

A linear regression analysis was performed of the three limit states to demonstrate the effect of shear wall amount in the high-rise RC buildings on the buildings’ resistance against seismic loads. Linear regression analysis shows the relationship between shear walls amount which are 0%, 23%, and 39% for buildings 1, 2, and 3, respectively and the fragility curves parameters such as mean and standard deviation of building performances in term of selected IM. In this study, only the mean of the sixteen ground motion parameters $S_a(T_{1,5\%})$ was selected to perform the linear regression analysis. Mean of the $S_a(T_{1,5\%})$ values was calculated after taking natural logarithmic ($\lambda = \ln(x)$) (where $x$ is ground motion parameters obtained from IDA curves as shown in second column of Table 6-3). The calculated mean values of the three limit states in both directions are shown in Table 6-4. Linear regression relationship between the mean of the ground motion parameters and the shear wall percentage is defined as:

$$\lambda = ay + b$$

Eq. 6-2

where,
$y = \text{the amount of shear wall used in the building in percentage (i.e. total shear wall length in plan / total length of walls in buildings, for example, total shear wall length is 136 ft and total building wall length is 606 ft. Therefore, the amount of shear wall is equal to 23-percentile).}$

$a & b = \text{the regression analysis coefficients}$

**Table 6-4 Fragility curve parameters of slight damage, moderate damage, and collapse of three buildings**

<table>
<thead>
<tr>
<th>Buildings</th>
<th>Shear wall amount, %</th>
<th>Direction</th>
<th>Mean of limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Slight damage</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Moderate damage</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Collapse</td>
</tr>
<tr>
<td>Building 1</td>
<td>0</td>
<td>X</td>
<td>-1.36</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-0.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-0.32</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Y</td>
<td>-1.19</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-0.60</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-0.33</td>
</tr>
<tr>
<td>Building 2</td>
<td>23</td>
<td>X</td>
<td>-1.39</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-0.36</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-0.23</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Y</td>
<td>-1.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-0.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-0.29</td>
</tr>
<tr>
<td>Building 3</td>
<td>39</td>
<td>X</td>
<td>-1.17</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Y</td>
<td>-1.07</td>
</tr>
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<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.41</td>
</tr>
</tbody>
</table>

R-squared ($R^2$) value was calculated for those regression lines which is a statistical measure of the percentage of fitting regression line to the given points. Linear line of 100% fitting to a given point is obtained when $R^2 = 1$. In this study, $R^2$ was determined for all regression lines to obtain the percentage of fitting. Linear regression analysis results are shown in Figure 6-20 and Figure 6-21 which include $R^2$ values and equations of $\lambda = ax + b$.
Figure 6-20 Shows linear regression analysis of mean of slight damage, moderate damage, and collapse limit states in *x-direction*

Figure 6-21 Shows linear regression analysis of mean of slight damage, moderate damage, and collapse limit states in *y-direction*
According to these results, building resistance against seismic loads is generally improved by increasing shear walls. For the slight damage level in the $x$-direction and $y$-direction (blue lines), $R^2 = 0.5009$ and $R^2 = 0.9872$, respectively. Different $R^2$ values between the $x$ and $y$-direction maybe be attributed to that building 2’s shear walls in the $y$-direction are longer than those in the $x$-direction (i.e. in $y$-direction four 20 ft shear walls are available, but in $x$-direction four 14 ft shear walls are placed.), resulting in much larger stiffness in the $y$-direction than the $x$-direction. So the improvement of seismic resistance in the $y$-direction is more obvious than the $x$-direction for building 2 at the slight damage limit state, which is highly dependent on the stiffness.

For the moderate damage limit state, $R^2 = 0.9906$ and $R^2 = 0.9805$ for $x$ and $y$-directions, respectively which means that buildings seismic performance at the moderate damage limit state linearly increased with the shear wall amount. Therefore, if designers want to have the best performance at the moderate damage limit state, increasing shear wall percentage might be an effective measure.

For the collapse limit state, $R^2 = 0.7623$ and $R^2 = 0.7008$ along the $x$ and $y$-direction, respectively. This result shows that generally increasing shear walls will improve building’s stability against earthquakes. However, $R^2$ being relatively far away from 1 means that shear wall location and building configuration may also play an important role at the collapse limit state that is not considered in this regression analysis, since this $R^2$ values is only a measurement between the shear wall percentage and the ground motion intensity required at the collapse limit state.

In general, seismic performances of high-rise RC buildings linearly increase with shear wall percentage especially at the slight damage and moderate damage limit states.
For the collapse limit state this is also true, but building configuration and shear wall locations will also greatly affect the building’s stability against collapse that cannot be ignored.

6.5 Conclusion

This chapter illustrates the multi-record IDA curves of the three high-rise RC buildings in both directions under sixteen ground motions and the DM and IM dispersions were calculated for those multi-record IDA curves. According to the dispersion results, buildings 2 and 3 have general smaller dispersion values than building 1 indicating the relative robust seismic responses of buildings 2 and 3 with added shear wall (stiffness) which is a preferred response by the designers. Three limit states were selected to construct the fragility curves which are slight damage, moderate damage, and collapse limit states. Two sets of fragility curves were developed: 1) three limit states of the same building and 2) the same limit state of three buildings. Finally, linear regression analysis was performed to show the effect of shear walls percentage on the buildings seismic responses in a statistical method. Developed fragility curves and the linear regression analysis both revealed that generally shear walls improve buildings’ seismic performance at the slight and moderate damage limit states with the increased stiffness. Shear wall percentage has the most effect on the moderate damage limit state as indicated by both the fragility curve and the regression analysis. However at the collapse limit state, shear wall location and configuration might also affect the building’s stability except for the shear wall percentage that requires further study.
7 Conclusion and Future Recommendation

Seismic fragility assessment of three reinforced concrete (RC) high-rise buildings, each 12-story high, were carried out and their performances were compared. First building is a Moment Resisting Frame (MRF), second is a MRF with shear walls on the exterior walls, and the third building consisted of most shear walls with only six exterior columns. Three buildings, located at Los Angeles, California, United States, were design using the STAAD Pro (2007) considering both gravity and lateral loads. Lateral loads included both wind and seismic loads and they were calculated and applied to the buildings following the American Society of Civil Engineers (ASCE 7) specifications. Specifically, buildings were designed to resist seismic forces calculated using the Equivalent Lateral Forces (ELF). Fragility curves were developed to show the probabilistic comparison of seismic responses among the three buildings in both x and y-directions using the results obtained from the Incremental Dynamic Analysis (IDA). To perform IDA of the three dimensional building models, sixteen ground motion pairs were selected based on the FEMA p695 criteria and scaled to match the design response spectrum suitable for the building location and to a spectral acceleration at the fundamental period with 5% damping $S_a(T_{1,5\%})$. Then, these sixteen ground motion pairs were applied orthogonally to the three dimensional building models and their IDA curves were obtained using the nonlinear time history analysis with increasing the ground motions’ intensity gradually until the building collapse. IDA curves were then created in terms of $S_a(T_{1,5\%})$ as the Intensity Measure (IM) and maximum inter-story drift ratio as the Damage Measure (DM). Slight damage, moderate damage, and collapse limit states were adopted to develop the fragility curves of the three buildings. Finally, linear regression analysis was performed to show the effects of shear walls in the
high-rise RC buildings’ on their seismic performances. Results showed that generally shear walls improve buildings’ seismic performance. However shear wall configuration also affects the seismic performance of high-rise RC buildings which needs further study to determine the optimal configurations based on specific performance target (i.e. the three limit states).

This chapter presents the major conclusions from this study and discusses recommendations for future work that will improve understandings of the high-rise RC buildings subjected to future ground motions.

7.1 Conclusions

The main objective of this study is to demonstrate the effect of shear wall on the seismic performance of high-rise RC buildings designed according to the codes. To obtain this objective, three RC high-rise buildings were designed, analyzed and their seismic responses were compared in a probabilistic method using the fragility curves. Three buildings have the same plan dimension and height, but different in plan configuration as described above. Seismic fragility curves were developed with respect to $S_a(T_{1.5\%})$ which were used for comparison. Three limit states (i.e. slight damage, moderate damage, and collapse) were identified for the three buildings and the main observations are summarized below:

1. From the DM and IM dispersion results obtained from the multi-record IDA curves, shear wall buildings have smaller dispersion than MRF buildings. It may be concluded that adding shear walls generally reduces dispersion to achieve increased robustness and reduce record-to-record variability which gives predictability in design.
2. Modal analysis confirmed that increasing the amount of shear wall in the building resulted in decreased fundamental period due to the increased stiffness (i.e. building 3 is stiffest and building 1 is weakest). Reasonable modal shapes were also obtained from the modal analysis. Both natural vibration properties (i.e. natural frequency and modal shape) revealed that the numerical model established in SeismoSturct is stable and rational that shall yield reasonable seismic responses from the nonlinear time history analysis with the proper nonlinear modeling (i.e. material nonlinearities, geometric nonlinearity) discussed in Chapter 4.

3. The fragility curves of the three buildings showed that: 1) At the slight damage limit state, buildings 1 and 2 have similar seismic performance and building 3 have higher performance (i.e. building 3 requires higher earthquake intensity to reach slight damage limit state) with small difference than the other two buildings. 2) At the moderate damage limit state, the fragility curves of three buildings are relatively equal displaced with building 3 being the rightest (best) and building 1 being the leftist (worst) illustrating that increasing shear wall effectively increase seismic performances at the moderate damage limit state. 3) At collapse limit state, building 3 is the most stable building with significant difference compared to building 1 and building 2 which have similar performance. Also, the linear regression analysis confirmed those observations from the fragility curves. In addition, the fragility curves and the maximum inter-story drift ratio of the collapse limit state showed that building 2 has minimal safety margin between reaching the moderate damage and the collapse limit state may be attributed to the irrational arrangement of shear walls at the external walls of the building.
In summary, shear walls increase buildings’ resistance against the seismic loads and decreases record-to-record variability which gives predictability design results. Also, locations and configurations of shear walls is essential to obtain desired seismic because buildings having both external and internal shear walls seems to be more stable with significant difference compared to MRF buildings and buildings having shear walls only in its parameter. Therefore, shear wall configuration is as important as shear wall amount in improving the seismic responses, and this is especially true at the collapse limit state.

7.2 Future Recommendations

More works can be done in future to improve the understanding of seismic performance of high-rise RC buildings and they are listed below.

1. In the nonlinear time history analysis to yield IDA curves, two horizontal ground motion components were applied to the three dimensional building models that were orthogonal to each other. To represent more realistic earthquake, vertical component should be added to this analysis.

2. This study only considered far-field ground motions and it is suggested that both far-field and near-field ground motions should be considered to yield models to all possible earthquakes’ input.

3. The inclusion of the effect of soil-structure interaction and site effect to the analysis requires further study to show their effects on the RC high-rise buildings’ fragility assessment.

4. For better comparison of seismic performance in a probabilistic way, it is highly recommended to use more ground motions to obtain more dependable fragility curves.
5. In this research, only the ground motion uncertainties were considered to develop fragility curves. More reliable results may be obtained by also considering material uncertainties as RC buildings have two different materials steel and concrete that show great variability in their mechanical properties that will affect the seismic performance of RC buildings.

6. To decrease dispersion, different intensity measure (IM) can be checked such as Peak Ground Acceleration (PGA). Researchers are still debating the effect of IM selection on the dispersion results.

7. One issue associated with IDA analysis is time consuming because IDA requires repeated nonlinear time history analysis. A recommendation of using parallel system especially for high-rise building is suggested by software programmers because high-rise buildings are complex buildings, composed of large number of structural members.

8. This research demonstrated that buildings having both external and internal shear walls are more stable, it is better to compare three buildings: 1) only internal shear wall, 2) both external and internal shears walls and; 3) only external shear walls. Because most buildings have internal shear wall columns which support elevator and stair cases.
References

ACI 318-11 (2011). Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, Farmington Hills, MI.


brief, US Department of Commerce, Building and Fire Research Laboratory, National Institute of Standards and Technology.


Vijayendra, K. V. (n.d.). *Earthquake resistant design of structures (Subject Code: 06CV834) Unit 5 & Unit 6: Seismic lateral force analysis*. Bangalore, India: Bangalore Institute of Technology.