ASSESSMENT OF DAMAGE INDEX FOR REINFORCED CONCRETE FRAMES SUBJECTED TO SEISMIC EXCITATIONS

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I hereby declare that all corrections and comments made by the supervisor(s) and examiner have been taken into consideration and rectified accordingly.

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ABSTRAK

Tujuan utama kajian ini ialah membentuk lengkung kerapuhan untuk rangka tetap berdasarkan perbezaan jenis ketinggian struktur, dan rekod pergerakan tanah. 3 set rangka konkrit dan keluli telah digunakan dalam kajian ini dengan ketinggian yang berbeza iaitu 3-, 6-, dan 9-tingkat untuk rangka tetap. Setiap struktur rangka direka berdasarkan Eurocode 2 dan 3 dengan bantuan Eurocode 8 untuk beban gempa bumi. Perisian SAP2000 dan ETABS telah digunakan sebagai perisian utama untuk analisa. Analisa pushover (POA) telah dijalankan untuk mendapatkan prestasi struktur berdasarkan beban statik. Daya corak segi tiga telah digunakan untuk menghasilkan hubungan dasar ricih-hanyut. Manakala, analisis dinamik tambahan (IDA) dijalankan dengan menggunakan tiga rekod pergerakan tanah. Keputusan daripada analisa IDA akan digunakan sebagai parameter utama untuk membentuk rangka kerapuhan. Lengkung IDA dibandingkan dengan 5 tahap prestasi seperti dinyatakan dalam kajian Xue et al. (2008) iaitu fasa operasi (OP), penghunian serta merta(IO), kawalan kerosakan (DC), keselamatan hayat (LS) dan runtuh pencegahan (CP). Daripada keputusan, ia telah membuktikan bahawa kerangka bertingkat rendah tidak sampai 1g PGA pada LS untuk Keadaan Kerosakan berbanding rangka pertengahan dan rangka tinggi. Berdasarkan hubungan antara Keadaan Kerosakan dan Indeks Kerosakan, rangka rendah dan rangka pertengahan mempunyai kerosakan kecil berbanding rangka tinggi yang menhadapi kerosakan dan perlu pembaikan. Tambahan pula, untuk Lengkung Kerapuhan, rangka bangunan tinggi menunjukkan kebarangkalian yang lebih tinggi untuk mencapai dan melebihi tahap prestasi berbanding bingkai pertengahan bertingkat.

ABSTRACT

In this study, the main objective is to develop fragility curve of regular moment-resisting frame based on different types of structural height, and ground motion records. 3 sets of concrete were used in this study and varied in terms of heights which are 3-, 6- and 9-storey for regular frame. Each structure frames was designed based on Eurocode 2 and 3 with the aid Eurocode 8 for earthquake loading. The SAP2000 and ETABS were used as the main tool to carry out the analysis. A pushover analysis (POA) was performed to get the performance of the structure due to static load. Triangular load was used to produce base shear-drift relationship. Then, an incremental dynamic analysis (IDA) was carried out with 3 ground motion records. While to develop the fragility curve, the result from IDA will be used as the main parameters. The IDA curves were compared with five level of performance level from Xue et al. (2008) study which are operation phase (OP), immediate occupancy (IO), damage control (DC), life safety (LS), and collapse prevention (CP). On the basis of the result of this thesis, it can be concluded that from POA result showed regular frames demonstrate a higher demand compared to irregular frames for concrete and steel frames. From the outcomes, it was proven that low-rise frame doesn’t achieve 1g of PGA at LS of limit state compared to mid and high-rise frame. Based on relationship between Damage Index and Damage State, the low-rise and mid-rise frame having minor damage compared to high-rise frame that having damage and should be repair. Furthermore, for the Fragility Curve, high-rise frame showed a higher probability of reaching and exceeding the performance level compared to mid-rise frame.

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LIST OF ABBREVIATIONS

DI	**D**amage **I**ndex
POA	**P**ushover **A**nalysis
IDA	**I**ncremental **D**ynamic **A**nalysis
MRCF	**M**oment **R**esisting **C**oncrete **F**rame
EC2	**E**uro**c**ode **2**
EC8	**E**uro**c**ode **8**
PGA	**P**eak **G**round **A**cceleration
BS	**B**ritish **S**tandard
MOSTI	**M**inistry **o**f **S**cience, **T**echnology and **I**nnovation
FEMA	**F**ederal **E**mergency **M**anagement **A**gency
PEER	**P**acific **E**arthquake **E**ngineering **R**esearch
NSA	**N**onlinear **S**tatic **A**nalysis
NDA	**N**onlinear **D**ynamic **A**nalysis
OP	**O**perational **P**hase
IO	**I**mmediate **O**ccupancy
LS	**L**ife **S**afety
CP	**C**ollapse **P**revention
DC	**D**amage **C**ontrol
DS	**D**amage **S**tate
RC	**R**einforced **C**oncrete

CHAPTER 1

INTRODUCTION

1.1 Background

Seismic weakness is basically characterized as "the proneness of some category of elements at risk to undergo adverse effects inflicted by potential earthquakes", i.e. the level of harm to the structure or of an area when the ground movement has been granted on the structure. Vulnerability is dependably specifically identified with hazard definition. Every structure is vulnerable to some extent on application of external loading. Several methods to evaluate the seismic vulnerability of structures have been proposed in recent years. One technique, the response-based damage index, has been critically evaluated for its applicability to seismic damage evaluation. This technique utilizes parameters, for example, such as base shear, stiffness, drift, rotation of an element, energy, and, in some cases, dynamic characteristics of a structure (mode shape and natural period of vibration) to calculate the state of damage using mathematical functions. Generally, damage index can be presented as an indicator for the damage of a whole structure or a structure component, which spoke to by an incentive in the vicinity of zero and one. Diverse sorts of damage index have been presented, which by and large prompt to various values because of their distinctive ideas.

There has been an earnest concern with respect to the basic reclamation of the current building class, which involves essential structures like historical buildings, lifeline structures and so forth. In this way, the earthquake engineers are always concentrating on the parts of creating probabilistic hazard models or the vulnerability models. Significant earthquakes throughout the last couple of decades demonstrate that seismic hazard is the main characteristic hazard that influences the socio-economic prospect of the society. So, only rough estimates can be made regarding the damage. In this way, the impact on any current building must be completed to check the sustainability of the structure. In urban regions of high seismicity, the probability of collapse is very high, requiring the idea of retrofit to a vast degree. One of the major parameter by which the seismic risk can be described is on the fragility of the structural component. At the point when the processed about fragility, the fundamental segment that should be considered is the uncertainty due to calculation method. Fragility curve is extremely basic to structural engineers, reliable specialists, and hospital and highway network. It is additionally utilized for damage assessment, assessment and improvement of seismic performance of structural and non-structural systems.

1.2 Problem Statement

The quantification of damage to reinforced concrete buildings due to earthquakes has utmost importance. This quantification helps in assessing seismic performance of the building through analytical methods and helps in several applications such as selecting retrofitting options. Seismic damage indices are widely used to predict possible damage. These damage indices have been formulated using response parameters of the structure that are obtained through analytical evaluation of structural response. The values of the damage index can be determine by comparing different damage levels of the analytical model with the corresponding damage states. The threshold values of damage indices also influence the shape of vulnerability curves. These values is really importance in taking decisions related to repair and retrofitting of the building. Hence, quantification of the relationship between damage index, which quantifies the damage using analytical models, and the damage states, which provide categorization of observed seismic damage is very important.

Despite the fact that, Malaysia not presented to coordinate but rather critical earthquakes damage from nearby short earthquakes experienced, for example, from Philippines and Indonesia yet having been influenced by both neighborhood and removed seismic tremors, come to understand that seismic hazard in Malaysia can possibly threaten the public safety and welfare which making harms properties concerning everyone. Ranau for example, has entered the phase of seismic activity. Thus Sabah especially, has to implement seismic (earthquake related) design for its structures and infrastructures as soon as possible to mitigate the safety and economic consequences due to seismic events and due to our local buildings are only designed for a vertical load, which are not resistant to earthquakes, making it susceptible to side-to-side movements (MOSTI 2015).

Since design codes that widely used in Malaysia are British Standard (BS) which not provide seismic design specification, support the fact that less than one percent of building in Malaysia are seismic resistant (Majid, 2009). Consequently, the most recent plan codes that utilized as a part of supplanting BS configuration codes is Standards Malaysia published the draft of "Malaysian Standard (MS) EN 1998-1 Eurocode 8: Design of structures for earthquake resistance – Part 1: General Rules, Seismic Actions and Rules for Building", in 2015. The research is to perform seismic vulnerability evaluation for Reinforced Concrete building based on different exhibitions by utilizing non-linear analysis.

1.3 Objective

1. To investigate the damage index of a reinforced concrete moment resisting frame by using modified Park and Ang damage model based on single ground motions records.
2. To develop the fragility curve for the reinforced concrete moment resisting frame.

1.4 Scope of Work

This research focus on the analytical model that used is uniform 3-, 6-, and 9-storey RC moment resisting frame. The research carry out the seismic vulnerability assessment of reinforced concrete building using pushover static and time history dynamic non-linear analysis using Eurocode 8 and Eurocode 2. The research will use previous recorded earthquake events data which is from Pacific Earthquake Engineering Research (PEER) NGA database website.

1.5 Organization of This Dissertation

This dissertation is divided into five main chapters including this "Introduction" in Chapter 1 that describes the background and problem statement of the research, the objectives to be achieved, research scope of work and the structure of the thesis. Chapter 2 presents literature review on research topics which are relevant to the proposed dissertation topics. Chapter 3 provides the methodology in study with the aid of flowchart description of the steps and flow charts of the study. Chapter 4 shows the analysis and discuss in detail about the research. Finally, Chapter 5 devoted to conclusions based on obtained result and suggestion for the future study.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

This chapter will reviewed and discussed of the research that related to the damage index of reinforced concrete building that subjected to the earthquake. Which are also related to the nonlinear analysis for static and dynamic analysis. Furthermore, damage state and fragility curve also will be reviewed.

2.2 Damage Index

The vulnerability of many existing structures may be due to structural weaknesses and low ductility. Common weaknesses in the structural system are due to incomplete load path, strength and stiffness discontinuities, vertical, horizontal and mass irregularities; weak column and strong beam, and eccentricities. Low ductility detailing is characterized as insufficient shear reinforcement, inadequate confinement, and insufficient anchorage length of the beam-reinforcement bars.

The state of damage of a component, a story, or the whole structure may be represented by an index. The damage index is used as an indicator to describe the state of the lateral load-carrying capacity and the reserve capacity of existing structures. Thus, the study on damage index and its availability is necessary.

Some damage indices are calculated for each component of the building (local damage index). The component damage indices may be integrated using a weighting procedure to provide the global damage index for the structure. These damage indices have been formulated using response parameters of the structure that are obtained through analytical evaluation of structural response. There are several techniques and approaches for damage analysis of structures, such as pushover analysis, nonlinear time history analysis, and vulnerability analysis. The typical response-based damage indices include ductility ratio, interstory drift, slope ratio, maximum drift, flexural damage ratio, low cycle fatigue, final softening index and Park-Ang index. The damage indices such as interstory drift and maximum drift, are fundamental and essential for representing the displacement or deformation.

Aghagholizadeh and Catbas (2015) characterize that, one of the test for civil engineers is to evaluate the amount of damage to a structure caused by moderate-to-strong earthquakes. An appraisal of the sort and amount of damage to a structure is required to decide its operability and resistance to aftershocks and future earthquakes. Damage estimation is vital undertaking in the fields of structural health monitoring, model updating, and decision making.

Krawinkler and Zohrei (1983) presented a damage index based on the prediction of fatigue life of the structural components subjected to a time history of deformation. The most well-known damage indices are calculated based on the energy absorption or the modal parameters including modal dynamic characteristics of the excited building. Modified Park-Ang damage index (Park et al., 1984; Valles et al., 1996) is one of the most popular combined damage indices, which is formed from linear combination of ductility and energy absorption capacity indices.

Aghagholizadeh and Massumi (2012) stated that all methods can be divided into two general categories, quantitative and qualitative. In this paper, a quantitative method is used to assess the state of the damage of structures. This paper introduces parameters as functions to calculate and express the state of damage to a structure as a numbered value. Valles et al. (1996) said that Park–Ang index was introduced in 1985 base on shear, stiffness, drift, rotation of an element, absorbed energy by elements, and Park–Ang damage indices were used to evaluate the degree and type of damage to a structure.

Wang et al. (2007) has been established a story damage index by using the modal frequency and mode shapes of the building, considering its condition both before and after the earthquake. Damage index relation has been derived based on the equation of motion of the structure. From simulation results, they came to the point that the presented damage index is more reliable comparing to the previous ones, which involved both mode shape and modal frequency. Furthermore, a mathematical expression has been developed to estimate the mode shape curvature of a damaged structure. The researchers found that the location of damage can be resulted directly in the change in the fundamental mode shape and its corresponding derivatives (Roy and Ray-Chaudhuri, 2013). Cosenza and Manfredi (2000) provided classification of the damage indices based on the type of analytical model used in calculating the damage index.

The study also evaluates the correlation between damage indices and damage states for assessing the condition of the structure and for decision making related to retrofitting and repairing of the damaged structure. The damage indices based on member-type model are classified as deformation-based damage indices; energy-based damage indices and combined damage indices. Borg and Rossetto (2010) used scoring system to rank available damage indices based on their abilities to quantify global damage and to identify critical damage location. The main objective of the scoring system was to select a few damage indices useful in repair and retrofitting decision making. Energy and deformation combined damage indices scored high in the ranking system and therefore only combined damage indices were evaluated with example buildings.

2.3 Earthquake Ground Motion

Ground motion records play a main role in establishing fragility curves. Selecting an appropriate ground motion and scaling the ground motions are very important in generating this curve. If the ground motion is randomly scaled up to a specific spectral acceleration, S_a, at period, T, over conservative structural response may occur (Baker et al., 2014)

The selected ground motion must come from previous recorded earthquake events. Ground motion can be selected from certain websites, such as Pacific Earthquake Engineering Research (PEER) NGA database website, Consortium of Organization for Strong Motion Observation System, or K-NET. Silva et al. (2014)

list other websites where ground motion records can be obtained, including the European Strong Motion database, the French Accelerometric Network, and the Swiss Earthquake Database.

A few parameters must be considered in selecting ground motion, including event magnitude, peak ground acceleration (PGA), distance, and soil type (Nazri and Alexander, 2012). In addition, ground motion characteristics must be considered to obtain accurate prediction and to minimize the dispersion of the analytical behavior of buildings. Ground motion characteristics that must be considered include ground motion intensity, spectral shape, duration, frequency content, near fault, amplitude, and number of cycle (Ibrahim and El-Shami, 2011; Ruiz-García and Negrete-Manriquez, 2011; Song et al., 2014).

2.4 Simulation Methods

To develop the fragility curve using the analytical method, a few popular simulation methods need to be applied. The assessment can be categorized into two main groups, namely, nonlinear static analysis (NSA) and nonlinear dynamic analysis (NDA). Table 2.1 shows some of software those used by past researchers.

Table 2.1 Available software used by researchers

Authors	Structural Type	Software
Singhal and Kiremidjian (1996)	MRCF	DRAIN-2DX
Akkar et al. (2005), Kumar et al. (2014), Hancilar et al. (2014)	MRCF	SAP2000
Lupoi et al. (2006), Ryu et al. (2011), Uma et al. (2011), Jeon et al. (2012), Réveillère et al. (2012), Shome et al. (2014) Silva et al. (2014), Hancilar et al. (2014),	MRCF (2D and 3D)	OpenSees
Ibrahim and El-Shami (2011)	MRCF	SeismoStruct

2.5 Nonlinear Analysis

Non-linear analysis perform better than linear analysis which non-linear analysis is much effective by providing realistic estimate of seismic demand of structural components deforming, strength and stiffness deterioration in the inelastic range. The area of large deformation demands and obtaining desired behavior are identified.

2.5.1 Pushover Static Analysis (POA)

Nonlinear static analysis or pushover analysis (POA) is one of the methods used to develop vulnerability seismic curve. Polese et al. (2013) initially evaluated the appropriateness of POA in damage analysis, from which they developed the fragility curve. They conducted the analysis for intact structures and damaged buildings, resulting in a capacity curve.

Moreover, Kumar et al. (2014) mentioned that capacity curve can represent mean or mean plus/minus with one/two/three times standard deviation capacity curves. From these capacity curves, the results can be compared with those of the PBSD in generating fragility curve.

2.5.2 Incremental Dynamic Analysis (IDA)

Choosing a nonlinear analysis tool and considering its limitation are important. This tool can give an accurate investigation and stable nonlinear time history analysis (NTHA) of the structure (Farsangi et al., 2014). The nonlinear dynamic analysis (NDA) or NTHA method considers geometric nonlinearity and material inelasticity in predicting the displacement behavior and collapse load. In addition, this method requires ground motion. A suitable set of ground motion is needed to ensure the accuracy of the fragility curves. However, the suitability of the set of ground motion is a significant issue (Billah and Alam, 2014).

Vona (2014) investigated a fragility curve based on different methods of analysis and two types of analyses, namely, POA and NDA. According to this study, NDA is the most accurate method in investigating the MRCF performance. This method can consider the real characteristics as inputs, from which it can evaluate structural response.

In addition, Silva et al. (2014) reported that NDA applies acceleration time history analysis, which then leads to accurate results. However, they found that NDA is time consuming. Thus, they introduced several methods, such as capacity spectrum method, displacement coefficient method (DCM) and N2 method, as alternatives. In conclusion, they suggested using NSA as a valid alternative for obtaining results rapidly and accurately.

Billah and Alam (2014) argued that NTHA requires a large number of ground motion, making the computational expensive. Thus, they introduced IDA to replace NTHA. They mentioned that Luco and Cornell (1998) first developed this method, which used to be a part of NTHA (both are found to be similar). However, ground motion in IDA scales incrementally, resulting in a different performance depending on the intensity level.

The previously mentioned suspicion is supported by Colapietro et al. (2014), who contended that IDA is an expansion strategy for NTHA or NDA. This strategy legitimately appraises the execution of structure under seismic load through specific arrangements of ground motion records, and scales the ground motion records to obtain the response curve. After contrasting the consequences of IDA and POA strategies, they inferred that POA indicates great connection with IDA. However, the previous is more moderate than the last mentioned, particularly in anticipating higher mode effects in the post elastic range, which considers irregular buildings for limited capabilities of fixed load distributions. IDA can be used to investigate complexities and extreme irregularities of analyzed buildings. Given that the reliability of an analysis is related to the level of knowledge, the authors suggest that destructive and non-destructive tests are to be performed to obtain more realistic estimations of seismic variability.

2.6 Damage State

In order to retrofit decision, it is necessary to quantify the structural damage. Therefore many damage models have been developed. Damage index is a mathematical model for quantitative description of the damage state of the structures and in most cases it has a correlation with the actual damage in earthquakes. There are various ways to categorize the damage indices. The simplest way is the correlation between damage indices and observed damage. For example Park et.al (6,9) classified the structural damage as None, Minor, Moderate, Severe and Collapse.

The damage states, with clear definition of the damage and failure mechanisms, allow users to evaluate post-earthquake status of buildings and also provide categorization of the damage for further use, such as for assessing seismic intensity. The damage states developed on the basis of cost-ratio or damage factor effectively link ground motion parameters such as the peak ground acceleration to structural and non-structural damage and consequently to the cost of damage; which are useful in estimating economic losses.

HAZUS (1999) used predefined set of cost ratios for buildings to forecast the damage and loss in buildings due to future earthquakes. On the other hand, FEMA 273 (1997) provided damage classification based on expected performance of structure in terms of building safety and serviceability after an earthquake. In order to correlate damage indices with the damage in actual buildings through damage states; the damage states should be defined with limiting values of measurable engineering parameters, capable of representing both global and local damage. The thresholds of the engineering parameters can be derived from experimental and/or observational studies. However, available damage states are based on damage factor, on engineering judgement or on experimental calibration using very limited data. The available damage states neither define damage states in terms of structural response parameters nor explicitly consider the differences in building lateral load resisting system and damage to nonstructural elements.

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2.7 Performance Based Seismic Design (PBSD)

Manafpour and Moghaddam (2014) reviewed the advantages and disadvantages of probabilistic PBSD by considering all its constraints and limitations. They found that PBSD provides quantitative measure for structural damage by considering specific earthquake level. PBSD can be used for several purposes:

i. Obtain better performance results for new buildings;
ii. Determine performance in accordance with code provisions with the subsequent development of the required adjustment;

iii. Enhance current provisions to obtain good designs; and

iv. Provide an efficient retrofit design procedure.

These authors argue that the performance of seismic assessment depends on two factors, namely, the ground motion types and resisting lateral load and height of the buildings. Meanwhile, PBSD can be determined based on the percentage of maximum interstorey drift. Interstorey drift was used because this factor can be easily measured during the analysis and provides clear result. Interstorey drift can be classified into five categories, namely, operational phase (OP), immediate occupancy (IO), damage control (DC), life safety (LS), and collapse prevention (CP) (Xue et al., 2008; Ibrahim and El-Shami, 2011). By contrast, other authors, such as Uma et al. (2011), classified interstorey drift into slight, moderate, extensive, complete, and collapse.

Ibrahim and El-Shami (2011) defined each limit state. The building is at an OP state when it is suitable for normal use with least or no damage. At an IO state, the building has minimal or no structural damage and minor non-structural damage. The LS state is when the building appears to have structural and non-structural damages, which require repairs before re-occupancy. At a CP state, the structural and non-structural parts of the building are prevented from collapsing and are not considered weaknesses of the structure.

A few guidelines, such as FEMA-356 and ATC-40, have been established to improve building performance (Charalambos et al., 2014). The PEER center methodology has been proposed to gain an overall assessment of building performance at any intensity level and limit state by integrating data related to seismic hazard and damage from the structural analysis and loss. Figure 2.1 shows limit state and ranges from FEMA-356 (2000) and summarize of limit state was presented in Table 2.2 .

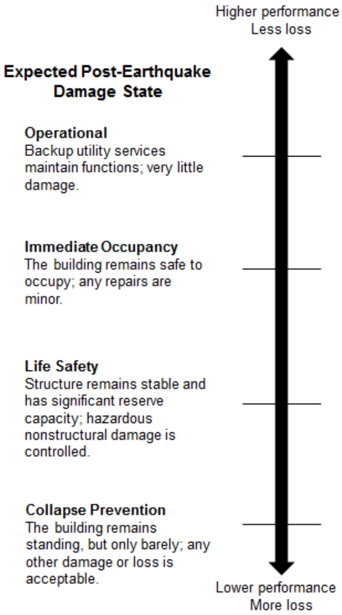

Figure 2.1 Limit state and ranges (FEMA-356, 2000)

Table 2.2 Summarize of limit state

Guidelines or Authors	ATC (1996)	Xue et al. (2008)	FEMA-356 (2000)	Ibrahim and El-Shami (2011)
Limit State	- Operational - Immediate Occupancy - Damage Control - Life Safety - Structural stability	- Operational - Immediate Occupancy - Damage Control - Life Safety - Collapse Prevention	- Operational - Immediate Occupancy - Life Safety - Collapse Prevention	- Operational - Immediate Occupancy - Damage Control - Life Safety - Collapse Prevention

2.8 Fragility Curve

Billah and Alam, (2014) defined fragility curve as the probability of reaching or exceeding a specific damage state under earthquake excitation. Seismic risk analysis can be improved through the dependent vulnerability of fragility curve. The general equation to develop fragility or conditional of probability is expressed as follows:

$$\text{Fragility}=P[\text{LS}|\text{IM}=y] \qquad \textbf{Eq. (1)}$$

where,

LS = limit state or damage state (DS);

IM = intensity measure (ground motion); and

y = realized condition of ground motion IM.

Various equations were derived by previous research. However, all the equations are based on Equation (1) which is a general equation for generating a fragility curve. Although most of these studies used different equations to generate their versions of the seismic vulnerability curve, some researchers (Yamaguchi and Yamazaki, 2000; Kirçil and Polat, 2006; Ibrahim and El-Shami, 2011) used similar equations in their studies. This equation, the simplest one in the group, is expressed in Equation (2). This equation is given as follows:

$$P(x)=\Phi\left(\frac{\ln X-\lambda}{\varsigma}\right) \qquad \textbf{Eq. (2)}$$

where,

$\Phi[.]$ = Standardize normal distribution

X = Lognormal distributed ground motion index

λ = Mean

ς = Standard deviation

The fragility curve is established to provide a prediction of potential damage during an earthquake. This curve represents the seismic risk assessment and is used as an indicator to identify the physical damage in the strongest mainshock. Apart from the mainshock, probability aftershock must also be investigated to decide whether or when to permit re-occupancy of a building. The fragility function is also directly used to prevent damage cost and loss of life during seismic event.

Two main components in the probabilistic seismic risk assessment have been identified. These components include an information ground motion hazard for the location of structure and fragility knowledge with respect to the intensity of ground motion. Polese et al. (2014) stated four important factors available for a large database, which include the number of storeys, age of construction, regularity (in plan, elevation, and in fill), and position of building in the block. Silva et al. (2014) proposed a vulnerability curve using the HAZUS tool (HAZUS, 1999) for risk assessment. The curve was created specifically for buildings in the US.

2.9 Summary

This chapter discusses the detail flow of the previous study starting from designing model, selecting and scaling the ground motions, simulation methods, performance based seismic design and the development of fragility curves. There are four types of method to develop fragility curve which are expert-based method, empirical method, analytical method and hybrid method. One of the simplest method and widely used is by using analytical method. It is more accurate method because it is considering all the uncertainties.

Therefore in this study, analytical method is applied. To carry out analytical method structural model of regular frames are generated and designed based on Eurocodes for low-, med- and high-rise buildings. Most of the prior study saying that, incremental dynamic analysis (IDA) is the most accurate method since this method considers ground motions as inputs. Selecting and scaling the ground motions were very important to get accurate result.

ETABS software is a main tool in order to perform the IDA. Five categories are used as indicators to the performance based seismic design (PBSD) which are operational phase (OP), immediate occupancy (IO), damage control (DC), life safety (LS), and collapse prevention (CP). Fragility equations that have been proposed by Ibrahim and El-Shami (2011) are be used because it is simple and incorporate the drift of the structure that can relate to the damage of the structure.

CHAPTER 3

METHODOLOGY

3.1 Overview

This chapter will discuss the methodology to investigate the damage index (DI) and to develop the fragility curve (FC) based on nonlinear analysis; pushover analysis (POA) and incremental dynamic analysis (IDA). Moment-resisting frame for concrete considered in this dissertation are design based on Eurocode 2 (concrete). Eurocode 8 is used for designing seismic effect on moment-resisting frame. Figure 3.1 shows the flow chart of methodology of analysis. The details of the methodology will be explained explicitly in this chapter.

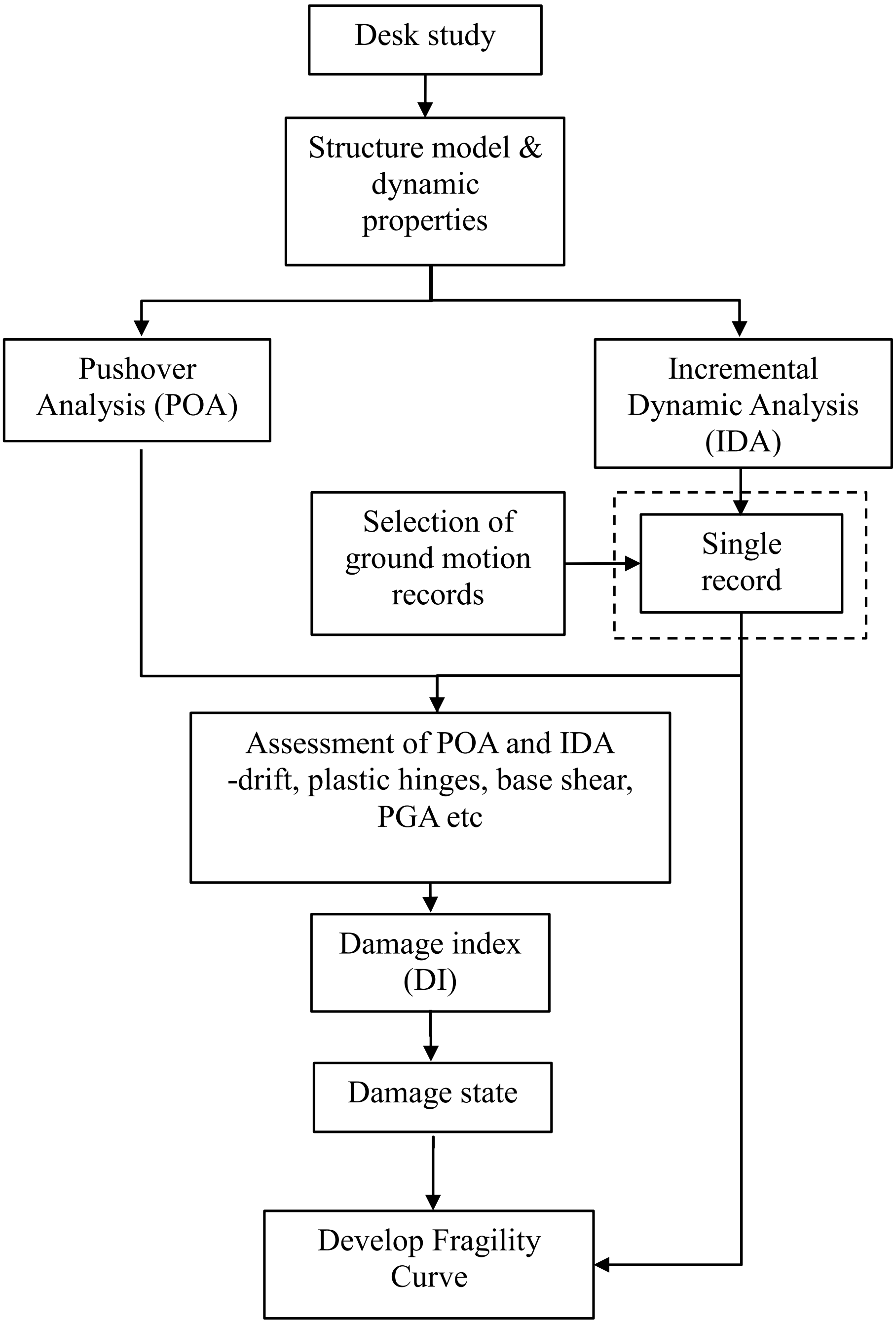

Figure 3.1 Flowchart of the Methodology

3.2 Selection of the structure model and dynamic properties

In this study, three set of model Moment-Resisting Frames (MRF) were analysed with different type of heights and place. These frames abide by the Eurocodes (EC). Each frame had 3 bays measuring 6 m each and identical height of 3 m for 3-, 6- and 9-storey regular frames. The materials used are concrete. Figure 3.2 shows the illustrated model for all storey height.

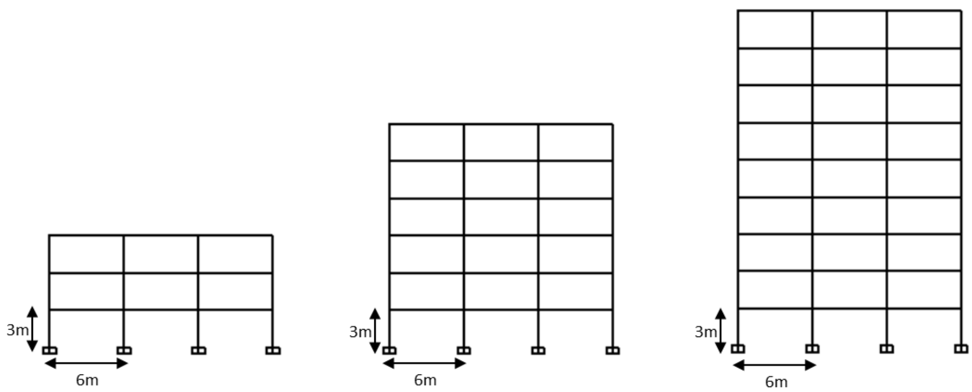

Figure 3.2 Regular MRCF

The structures will used soil class A with peak ground acceleration (PGA), a_{gr}, was assumed 0.5 g or 5 m/s^2. Based on EC8 (BSI, 2004), type soil A is rock or other rock like geological formation with at least 5 m of weaker material at the surface. Thus, to avoid the soil-structure interaction in the analysis, soil class A will be used. The importance value used was 1 and the behaviour factor, q was 4 for regular moment resisting frame with medium ductility class (DCM).

The designs for MRCF were based on existing building by using EC2 (BSI, 2004a) and EC8 (BSI, 2004) standards. Several assumptions were made during the design of MRCF. Compressive stress of concrete was 30 N/mm^2 and yield stress of reinforcing steel was 460 N/mm^2. Table 3.1 shows the size of beam and column for regular MRCF , respectively.

Table 3.1 Size of beam and column for regular MRCF

No of storey	3-Storey		6-Storey		9-Storey	
Section	Beam	Column	Beam	Column	Beam	Column
Size (mm)	350 x 500	500 x 500	350 x 500	500 x 500	350 x 500	500 x 500
Reinforcement	5T16	5T32	5T16	5T32	5T16	6T32
Shear Link	8 mm @ 150 c/c		8 mm @ 150 c/c		8 mm @ 150 c/c	

All frames were imposed by the dead, live and lateral loads. The lateral loads were design based on EC8. The self-weight of the structures, weight of the permanent partition such as finishes, brick wall, and all permanent construction are under dead load effect. The details of dead and live loads are as follow.

By assuming concrete density = 24 kN/m^3

The area of slab = 36 m^2 (6 m x 6 m)

Thus, the self-weight of the slab = concrete density x slab thickness (0.15 m)

= 3.6 kN/m^2

Table 3.2 shows the loads considered as the dead load and the live load are tabulated in Table 3.3