EFFECT OF REPEATED FAR FIELD EARTHQUAKE ON THE DUCTILITY DEMAND OF LOW-RISE REINFORCED CONCRETE BUILDINGS

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UNIVERSITI SAINS MALAYSIA

2011

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by

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Thesis submitted in fulfilment of the

requirements for the degree of

Master of Science (Structural Engineering)

December 2011

ACKNOWLEDGEMENT

On the completion of this thesis, I would like to express my highest gratitude to Allah S.W.T., for His love and mercy. Praise and peace upon Prophet Muhammad S.A.W, his family and his companions.

Firstly, my appreciation goes to my supervisor Assoc. Prof. Dr.Taksiah A.Majid for her guidance and help during completing this project. Moreover, she’s also willing to sacrifice her busy schedule as lecturer to spend time with her students to discuss problems during this project. Without her help, I won’t be able to finish this project. In addition, I also would like to express my gratitude to Mr Ade Faisal, PhD student of Civil Engineering School, USM as my mentor for helping and advising in completing this dissertation. He help’s a lot from beginning until this project finish.

I also like to acknowledge and extend my gratitude to the financial support from USM fellowship scheme and Research Grant (100/PAWAM/814115) during completion of this thesis. A high appreciation also addressed to staffs School of Civil Engineering, Universiti Sains Malaysia .

Lastly, special thanks to my mother, Pn. Minah bt Hussin, family and friends who give endless support to me while facing hard time completing this project. Also thanks to my friends who are in the earthquake group, i.e. Irwan, Zulham and Sabila for the moral support. I really appreciate what they have done for me. Thank you very much.

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LIST OF ABBREVIATION

EC 8	Eurocode 8
FEMA	Federal Emergency Management Agency
FFE	Far Field Earthquake
MDOF	Multi Degree of Freedom
NEHPR	National Earthquake Hazards Reduction Program
NFE	Near Field Earthquake
NTHA	Non Linear Time History Analysis
SDOF	Single Degree of Freedom
UBC	Uniform Building Code
USGS	U.S. Geological Survey

LIST OF SYMBOLS

Δ	Displacement
R	Force Reduction Factor
μ	Ductility
V	Shear
Ω_d	Overstrength
μ_δ	Translation Ductility Factors
γ	Base Shear Coefficient
T_1	Fundamental Period
W	Weight
r	Post Yielding
α	Unloading Stiffness Parameter
β	Reloading Stiffness Parameter
θ_p	Rotation capacity
K_e	Component Stiffness
M_y	Yielding Moment
M_c	Capping Strength
θ_{pc}	Post-Capping Deformation Capacity
H_i	Column Height
N	Number of Storey
T_c	Upper Limit
δ	Ratio of the lateral stiffness
L_b	Beam length
L_c	Column length
λ	Correction factor
a_g	Peak ground acceleration
χ_p	Plastic curvature
L_p	Length of plastic hinge
θ_y	Yield rotation
K_0	Elastic rotation stiffness
Δ_y	Yield displacement

Δ_{max}	Maximum displacement
V_e	Maximum base shear
V	Base shear
V_i	Storey lateral load
M_b	Moments of resistance of the beam framing the joint
M_{col}	Moments of resistance of the columns framing the joint

KESAN GEMPA BUMI JARAK JAUH YANG BERULANG TERHADAP PERMINTAAN KEMULURAN BANGUNAN RENDAH KONKRIT BERTETULANG

ABSTRAK

Impak yang dihasilkan oleh gempa bumi pada bangunan harus diambil kira dengan serius untuk menyediakan bangunan yang selamat. Sebagaimana yang diketahui, fenomena gempa tidak berlaku secara bersendirian tetapi ia adalah fenomena yang berlaku secara berulang. Walaubagaimanapun, sehingga kini kod rekabentuk seismik mengabaikan kesan gempa berulang. Semasa gempa berlaku, antara faktor yang berkaitan dengan kerosakan bangunan dikenali sebagai permintaan kemuluran. Tujuan kajian ini ialah menentukan kesan gempa bumi berulang terhadap permintaan kemuluran bangunan rendah konkrit bertetulang. Analisis sejarah masa tidak linear menggunakan perisian RUAUMOKO telah diaplikasikan untuk menentukan permintaan kemuluran bagi dua model bangunan rendah iaitu 3 tingkat dan 6 tingkat. Sebanyak 20 pasangan pergerakan tanah jarak jauh yang digabung secara rawak telah dilaksanakan dengan jumlah 1800 analisis. Hasil daripada kajian ini, terbukti bahawa kejadian gempa bumi berulang memberikan sebanyak 14.91% dan 48.69% kenaikan permintaan kemuluran bagi model bangunan 3 tingkat dan 6 tingkat. Sementara itu, faktor pengurangan daya juga memberikan kenaikan permintaan kemuluran sebanyak 113.39% dan 40.49% bagi model bangunan 3 dan 6 tingkat.

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ABSTRACT

Impacts of seismic activity on building should be considered seriously to provide a safe building. Earthquake phenomenon is not a single event but repeated phenomenon. However, current seismic design codes are ignored the effects of repeated earthquake. During the earthquake hits the ground, the factor that directly related to the damage of the structure is well known as ductility demand. The objective of this study is to determine the effect of repeated earthquake on the ductility demand of low-rise RC buildings. The nonlinear time history analysis performed to determine the ductility demand for two low rise building models which is 3 storey and 6 storey using RUAUMOKO software. There are 20 pairs of far field earthquake (FFE) randomly combined was used to performed 1800 analyses. Findings from this study showed that, the repeated earthquake give the increment 14.91% and 48.69% of ductility demands for 3 and 6 storey models, respectively. Meanwhile, force reduction factor also give the increment 113.39% and 40.49% of ductility demand for 3 and 6 storey models, respectively.

CHAPTER 1

INTRODUCTION

1.1 Background

Earthquake causes movement and ground shaking and consequently causes structural building to undergo displacement where it will be shifted quickly from it's original position due to the sudden force (seismic force).

Current studies show that seismic activity in Malaysia region had increased after the giant earthquake December 26, 2004 in Sumatran region which is the closest seismic region to Malaysia. The Sumatra earthquake (magnitude 7.2) that occurred on May 9, 2010 was also felt in several areas in Peninsular Malaysia, even though Malaysia is not in a high seismic zone but it is surrounded by countries that are in high seismic areas. Thus Malaysia can feel the vibrations as well. Table 1.1 shows the earthquake record in Malaysia since 1909 until July 2010.

However, Malaysia is not exactly located in the seismic region, but the effect of earthquake still can felt in several areas in Peninsular Malaysia. This phenomenon is caused by FFE, which is recorded within a few kilometres of the fault rupture. Even though the effect of FFE is not serious compared to near field earthquake (NFE), this type of ground motion also has significant effect on building performance.

Earthquake does not occur in single event but it comes with multiple events. This study tries to determine the influence of the repeated phenomenon on ductility demand for low rise building especially the effect by FFE. Therefore, this study is significant for Malaysia scenario.

Table 1.1: Earthquake Intensity in Malaysia
(http://www.met.gov.my/index.php?option=com_frontpage&Itemid=1)

State	Frequencies	Maximum Intensity Observed (Modified Mercalli Scale)
Peninsular Malaysia (1909 - July 2010)		
Perlis	3	V
Kedah	18	V
Penang	41	VI
Perak	24	VI
Selangor	50	VI
Negeri Sembilan	14	V
Malacca	19	V
Johor	32	VI
Pahang	35	III
Terengganu	2	IV
Kelantan	3	IV
Kuala Lumpur / Putrajaya	38	VI
Sabah (1897 - July 2010)		
Sabah	40	VII
Sarawak (1874 - July 2010)		
Sarawak	17	VI

Source: *Malaysian Meteorology Department*.

1.2 Problem Statement

The effect of earthquake in Peninsular Malaysia, especially to the buildings on soft soil are occasionally subjected to tremors due to far-field effects (FFE- recorded within a few kilometres of the fault rupture) of earthquake in Sumatra (Nik Azizan, 2010). The seismic waves, generated from an earthquake in Sumatra, travel long distance before they reach Peninsular Malaysia bedrock. The mechanism of the (FFE) is illustrated in Figure 1.1.

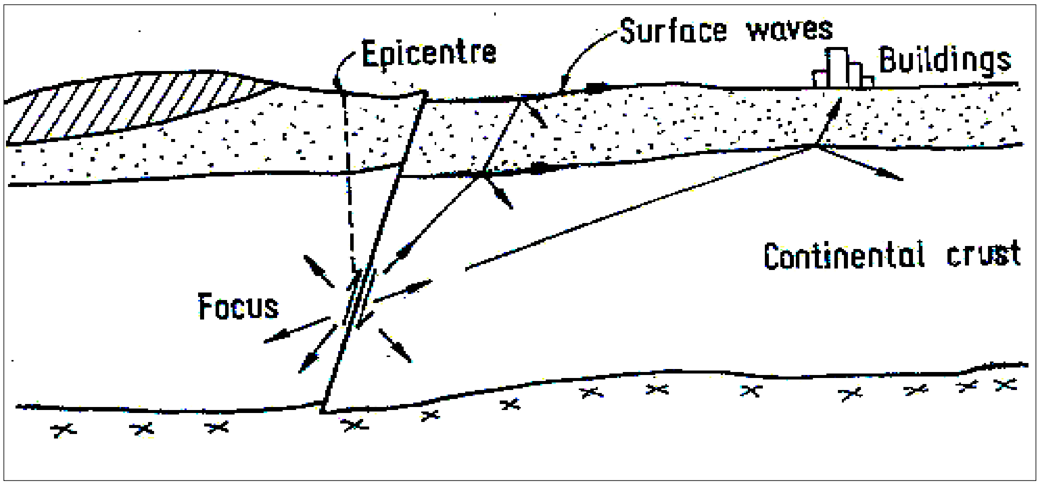


Figure 1.1: Mechanism of far-field effects of earthquakes (Balendra and Li, 2008).

The high frequency earthquake waves damped out rapidly in the propagation while the low frequency or long period waves are more robust to energy dissipation and as a result they travel long distances. When long period seismic waves reach the bedrock of Peninsular Malaysia, they are significantly amplified due to the resonance. Resonance is produced when they propagate upward through the soft soil sites with a period close to the predominant period of the seismic waves. The amplified waves cause resonance in buildings with a natural period close to the period of the site, and the resulting motions of buildings are large enough to be felt by the residence (Balendra and Li, 2008).

According to the definition of low rise building by Emporis Standard (2011), a low-rise building is an enclosed structure whose architectural height is below 35 meters, and which is divided at regular intervals into occupiable levels. It encompasses all regular multi-story buildings which are enclosed, which are below the height of a high-rise, and which are not entirely underground. Almost all the buildings in Malaysia can be categorized as low rise building because the height between 3 to 6 storeys and the effects of earthquake are significant to these types of the building for example houses, office, school and many more. Thus, tremors from the Sumatran earthquakes had brought safety concerns to the publics, government authorities, engineers and researchers especially when no earthquake design had been taken into practices in Malaysia (Adnan and Suradi, 2008, Adnan et al., 2006).

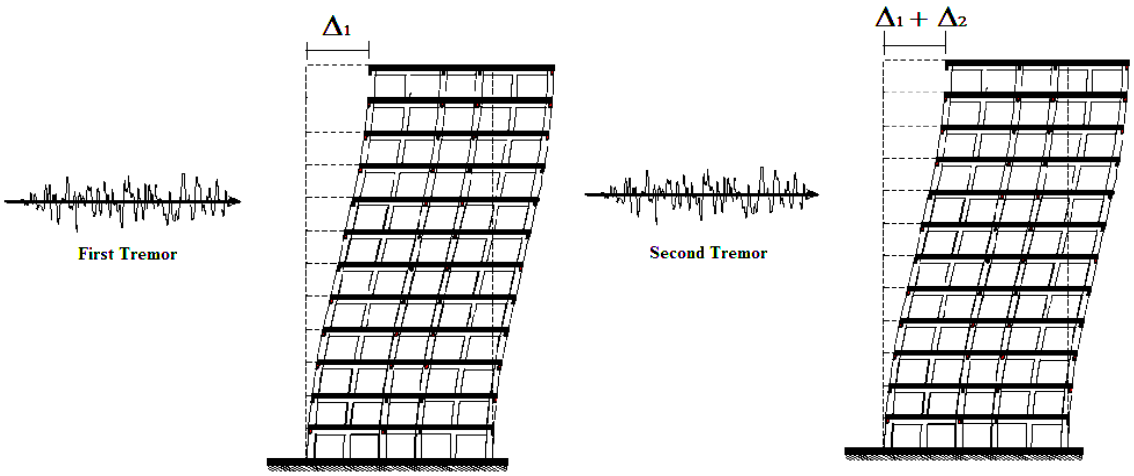
Therefore, should any earthquake occur, the damage or collapse not only effect general commercial buildings, but also public-service buildings such as police offices, communication centres and hospitals would result in very large life and economic losses as well as cause critical interference with the function of the nation.

Most structures were designed according to current code provisions which will sustain damage in the event of a design-level earthquake even if they perform exactly as expected. It is well known ductility demand is directly related to structural damage. The relationship between ductility demand and structural damage is very important for structural performance evaluation (Hatzigeorgiou, 2010a).

Earthquake phenomenon does not occur in single event, but earthquake is a repeated phenomenon. There are could be more than two tremors after the first tremor hits the ground. However, very few studies have been reported in the literature regarding the repeated earthquake phenomenon and this phenomenon is ignored in the 'earthquake design' (Hatzigeorgiou, 2010a; Hatzigeorgiou, 2010b; Hatzigeorgiou and Liolios, 2010; Hatzigeorgiou and Beskos, 2009). Hatzigeorgiou and Liolios (2010) noted that the sequences of ground motion have a significant effect on the response and hence, on the design of the reinforced concrete frames.

Figure 1.2 shows the effect of sequences of the ground motion. It is well known that the inelastic flexible system present permanent displacement for single strong earthquake. For any other incoming ground motion, permanent displacements are obviously cumulated and therefore the maximum displacement appears to be increased (Hatzigeorgiou, 2010a). After the first tremor hits the ground, the building will have displacement, Δ_1. The displacement, Δ_1 will increase when second tremor comes and contribute second displacement, Δ_2.

The damages of the structure are directly related to the ductility demand of the building (Hatzigeorgiou, 2010b). Therefore evaluation of their relationship is very important for structural performance. Ductility demand required by multiple earthquakes is notably higher than that required by single event (Hatzigeorgiou, 2010c). Equivalently, multiple seismic ground motions drastically reduce the corresponding force reduction factor for a specific ductility demand.



(a) (b)

Figure 1.2: Effect of sequence of the ground motion; (a) First tremor, (b) Second tremor (Hatzigeorgiou, 2010a).

1.3 Objective

Objectives of this study are:

i. To determine the effect of repeated earthquake on the ductility demand of low rise reinforced concrete building.

ii. To determine the effect of force reduction factor, R on the ductility demand of the low rise reinforced concrete building.

1.4 Scope of work

This study covered and focused in the following aspect:

i. Two generic RC models three storey and six storeys adopted from (Zarein and Krawinkler, 2009).

ii. Ground motion type considered in this study is FFE.

iii. 20 numbers of ground motion with 3 types of combination.

   a) Case 1: Single ground motion (main shock).

   b) Case 2: Repeated ground motion (fore shock – main shock).

   c) Case 3: Repeated ground motion (fore shock – main shock – after shock).

iv. Response parameter considered in this study is a ductility demand.

v. Five force reduction factors R= 1, 1.5,2,4 and 6 as recommended by Ruiz-Garcia and Miranda (2006) and with some modification as presented by Ade Faisal (2011).

1.5 Thesis outline

Chapter 1 covers the introduction of this thesis as well as the objectives of the thesis. Furthermore, this chapter provides the objective, scope of work and problem statement.

Chapter 2 covers all aspects which are involve in this study and literature review. This chapter is explaining about repeated earthquake phenomenon, force reduction factor, rotation capacity and the analysis that used in this study.

Chapter 3 explains the description of the model used in this study and the ground motion sequence used to analyze the model. In this chapter also briefly explain step – by – step procedures.

Chapter 4 is discussing the result of the study. This topic covers the effect of repeated FFE to the inter-storey ductility demand. Besides that, this chapter also covers the discussion about effect R and effect of fundamental period to the inter-story ductility demand.

Chapter 5 provides the conclusion for this study and recommendation for future study.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Earthquake causes movement and ground shaking and consequently causes structural building to undergo displacement where it will be shifted quickly from its original position due to the sudden force (seismic force). Generally, earthquakes cause significant damage within short distances of a fault and the effects of high frequency components of an earthquake have often been a subject of study in earthquake engineering.

In this chapter, some of terminology should be highlighted for better understanding in this study such as repeated earthquake, ductility, force reduction factor, R and ground motions.

2.2 Repeated earthquake phenomenon

The earthquake may occur repeatedly and it is difficult to predict the frequency of the earthquake hits the ground (Ellen, 2000). This phenomenon is very dangerous to the building in term of building performances. A few studies in repeated ground motion have been done recently. However, the influence of repeated earthquake is ignored in the code. After the ground shaking, the first wave of earthquake will hit the building and caused certain displacement to occur. For any other incoming ground motion, permanent displacements are obviously cumulated and therefore the maximum displacement appears to be increased (Hatzigeorgiou, 2010a).

In such cases, the structure already damaged after the first earthquake ground motion and not yet repaired, may become completely inadequate at the end of the seismic sequence. This accumulation of damage depends on the type of hysteretic structural behaviour and on the characteristics of the seismic events (Amadio et al., 2003).

2.2.1 Foreshocks, Main shocks and Aftershocks

The repeated phenomenon of earthquake basically consists of sequences which are known as foreshocks, main shocks and aftershocks. The largest quake in a sequence is the main shocks, occurring between any foreshocks and aftershocks. Foreshocks are smaller earthquakes that come before the bigger quake and not all main shocks have foreshocks.

For example, Table 2.1 shows the detailed of the three earthquake occurred at virtually the same location (8 km of Watsonville) and within 7 minutes of each other on May 9, 2000. The comparison of foreshocks, main shocks and aftershocks was plotted in Figure 2.1.

In this figure, main shocks have larger magnitude compared to foreshocks and aftershocks. For example, in the Northridge earthquake the main shock which is, the largest, had moment magnitude of 6.7. There were no foreshocks, but immediately after the main shocks and continuing for about five years there were more than 14,000 aftershocks. Thirty-six percent of the aftershocks occurred in the first month, which is typical (Ellen, 2000). Aftershocks usually have an orderly and steady rate of decay which means that they become less frequent with time. This does not mean that aftershocks necessarily decrease in magnitude with time.

Table 2.1: The detailed of the three earthquakes (USGS 2011).

Time, PDT	Magnitude	Latitude	Longitude	Depth	Designation
00:59:06	M=1.7	36.939	-121.679	8	Foreshock
01:00:55	M=3.3	36.246	-120.821	8	Main shock
01:06:02	M=2.9	36.244	-120.829	8	Aftershock

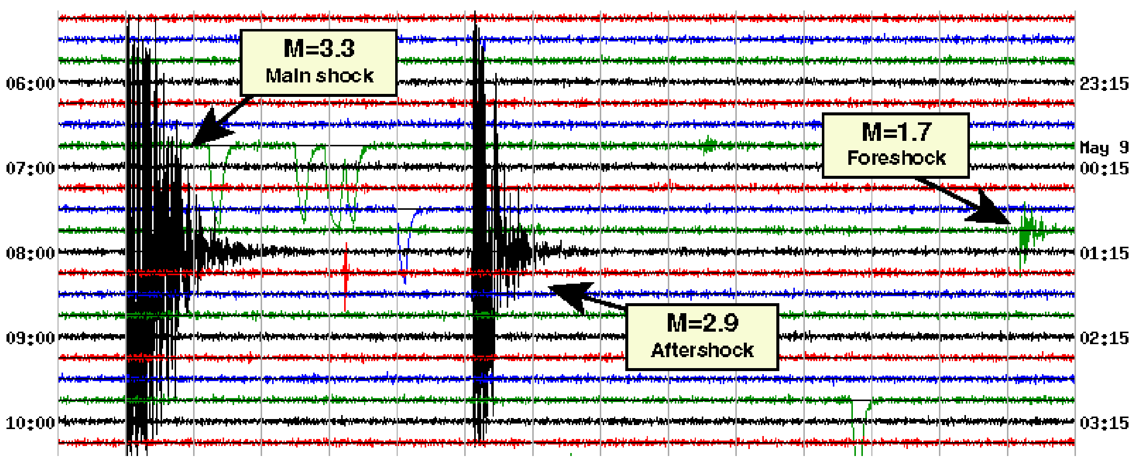


Figure 2.1: Comparison between foreshocks, main shocks and aftershocks (USGS, 2011).

Time is an important factor with aftershocks because and there are could be many aftershocks within the first hour or maybe a day, weeks, month even a year of the earthquake and aftershock decrease proportionately to the time since the main shock happened and the bigger earthquakes have more and larger aftershocks (Hubpages, 2011).

The bigger the main shock is the bigger aftershock will be, on average. The difference in magnitude between the main shock and largest aftershock ranges from 0.1 to 3 or more, but averages 1.2 (USGS, 2011). There are more small scale aftershocks than large ones. Aftershocks of all magnitudes decrease at the same rate, but because the large aftershocks are already less frequent, the decay can be noticed more quickly. Large aftershocks can occur months or even years after the main shock.

2.2.2 Effect of seismic sequence on ductility

Ductility demand required by multiple seismic ground motion can be notably higher than that required by single event. Equivalently, multiple seismic ground motions drastically reduce the corresponding force reduction factor R, for a specific ductility demand. In such case, the structure may become totally inadequate at the seismic sequence caused by damaged during the first seismic event.

According to Hatzigeorgiou (2010c), the multiple earthquakes lead to increase in ductility demands of two times or more the maximum single events value. The required ductility demand increased due to multiplicity of earthquake. The inelastic flexible systems will present the permanent displacement for single strong earthquake. For any other oncoming ground motion, permanent displacements are obviously cumulated and therefore the maximum displacements appear to be increased (Hatzigeorgiou and Liolios, 2010).

The accumulation of damage depends on the type of hysteretic structural behaviour (Amadio et al., 2003). According to FEMA P404A (2009), there are several type of hysteretic structural behaviour proposed over the year in estimating the seismic response of the structure such as elasto plastic behaviour, strength hardening behaviour, stiffness degrading behaviour and many more. The period of structure have great influence on force reduction factor R, while the earthquake magnitude and epicentre distance are significant factors. The total ductility demands, and therefore the cumulative damage levels can be controlled using appropriate force reduction factor, R.

2.2.3 Effect of seismic sequence on low rise building

Design for earthquake ground motion is often regarded as uneconomical, inappropriate, or too complex for low rise buildings, especially for areas in Malaysia which is within the low seismicity zone. In some cases, static wind pressures are found to govern the design and are assumed to be a suitable replacement for earthquake induced inertial forces.

However, recently many low rise RC building have suffered moderate to severe damage of structural and non-structural components in earthquakes (Tsai et al., 2000) and also the weakness in design and construction management.

2.3 Ductility Demand

The most important relationship in assessment of structural performance is a relationship between structural damage and the ductility demands (Hatzigeorgiou, 2010b). Ductility is defined as the ability of a material, component, connection or structure to undergo inelastic deformations with acceptable stiffness and strength reduction (Elnashai and Sarno, 2008). In seismic design, high available ductility is essential to ensure the plastic redistribution of actions among structural components and to allow for large absorption and dissipation of earthquake input energy (Razak, 2010).

Ductile systems may withstand extensive structural damage without collapsing or endangering life safety. Figure 2.2 compares the structural response of brittle and ductile systems. In the figure, curves A and B express force – displacement relationships for systems with the same stiffness and strength but distinct post - peak (inelastic) behaviour. Curve A is representing brittle systems. The brittle systems fail after reaching their strength limit at very low inelastic deformations. Meanwhile the V_{max} shows the maximum resistance for the system. The collapse of brittle systems occurs suddenly beyond the maximum resistance, because of lack of ductility.

Conversely, curve B corresponds to large inelastic deformations, which are typical of ductile systems. Whereas the two response curves are identical up to the maximum resistance V_{max}, they should be treated differently under seismic loads. The ultimate deformations Δ_u corresponding to load level V_u are higher in curve B with respect to curve A, i.e. $\Delta_{u,B} >> \Delta_{u,A}$ (Elnashai and Sarno, 2008).

The use of ductility factors permits the maximum deformation to be expressed in non-dimensional terms as indices of post-elastic deformation for design and analysis. Ductility factors have been commonly expressed in terms of the various parameters related to deformation, i.e. displacements, rotations, curvatures and strains.

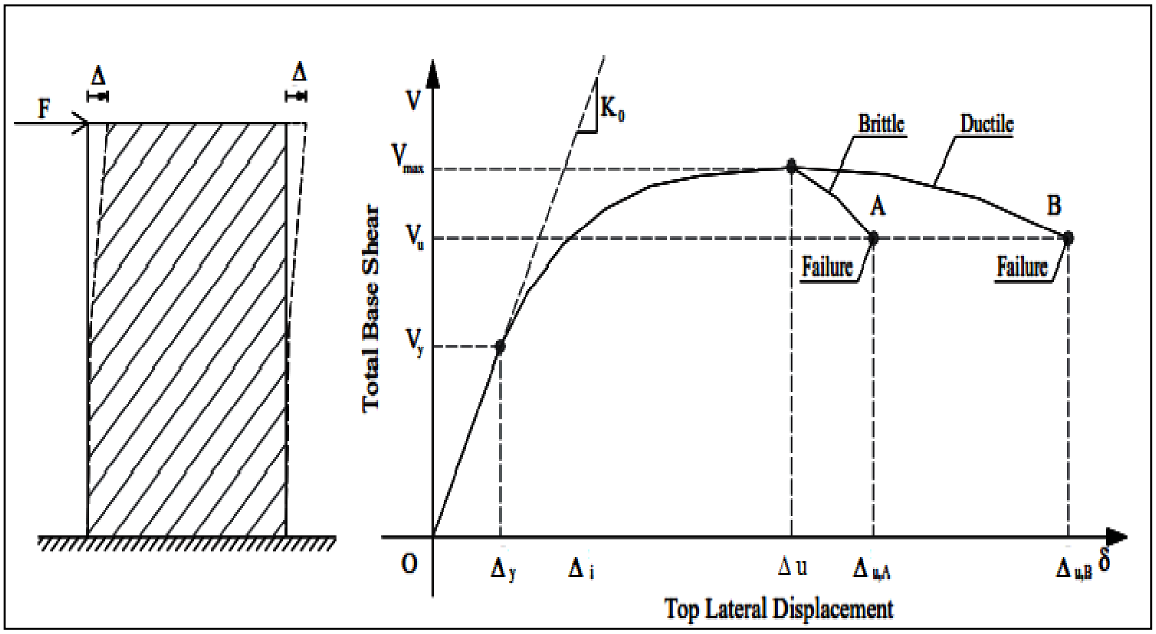


Figure 2.2: Definition of structural ductility (Elnashai and Sarno, 2008).

The required ductility of a structure, element or section can be expressed in term of the maximum imposed deformation. Often it is convenient to express the maximum deformation in terms of ductility factor. Elnashai and Sarno (2008) defined the ductility factor is the maximum deformation divided by the corresponding deformation present when yielding first occurs. Thus the ductility factor can be simplified as:

$$\mu = \Delta_{max} / \Delta_Y \tag{2.1}$$

Where,

Δ_{Max} is the maximum deformation

Δ_Y is the deformation present when yielding first occurs.

2.4 Force Reduction factor

The concepts of inelastic spectrum for specifying design action (forces) to be used for elastic analysis of structures have long relied in the seismic code. The distribution of forces resulting from such analyses often bears little resemblance to the expected during the actual earthquake. The concept of dividing the elastic response spectrum by a single factor to arrive at the (inelastic) design spectrum is a practical one and has been adopted by most seismic codes, including among others, the European Code (Eurocode 8, or EC8), and the American Codes (Uniform Building Code, UBC) and NEHPR(Kappos, 1999). The factor used for reducing the elastic response spectrum is called force reduction factor, R.

Elnashai and Sarno (2008) defined the force reduction factor as the ratio between elastic base shear, V_e and seismic design shears, V_d.

$$R = V_e/V_d \tag{2.2}$$

where V_e is an elastic base shear and V_d is the seismic design shear. There are many codes for force reduction factor; R and the numerical value are notably varied between seismic codes. For instance, the EC8 force reduction factor R, (also known as behaviour factor, q) ranges between 1.5 and 5.0 for RC frame structures. Meanwhile for US codes may be as higher as 8.0.

Force reduction factor are related to strength, ductility, over strength, and damping characteristic of the structure. Consequently, the force reduction factors are often expressed as a function of the system resistance, over strength Ω_d and translation ductility μ_δ factors (Elnashai and Sarno, 2008). There are varies factors that affect the over strength factor including height of the building, gravity loads, fundamental period, seismic intensity level, the structural system and the ductility level that used in design (Elnashai and Mwafy, 2002).

Elnashai and Mwafy (2002) noted that medium rise buildings exhibit lower over strength compared with low rise building. Therefore the minimum over strength of 2.0 can also be applied to the low rise building. Moreover, seismic forces generally play a less important role in the determination of cross-sectional sizes and reinforcements than do gravity loads, which govern the design of those buildings.

Figure 2.3 shows the typical global structural response of a building structure. The strength of the building is represented by the vertical axis. The strength is expressed in term of the maximum base shear while the horizontal axis represents the maximum displacement at a reference point (usually at roof).

In the Figure 2.3, Δ_{max} is the maximum displacement, V_e the maximum base shear in the elastic range, V_y the maximum inelastic base shear, V_d the base shear which corresponds to the occurrence of the first plastic hinge, and Δ_y the yield displacement of an elastic perfectly plastic equivalent system with a value corresponding to V_y and such that the areas under the actual and the idealized nonlinear response curves up to Δ_{max} are equal (Elnashai and Sarno, 2008). Then, the definition of all the basic terms pertinent to the force reduction factor is straightforward (Karavasilis et al., 2007).

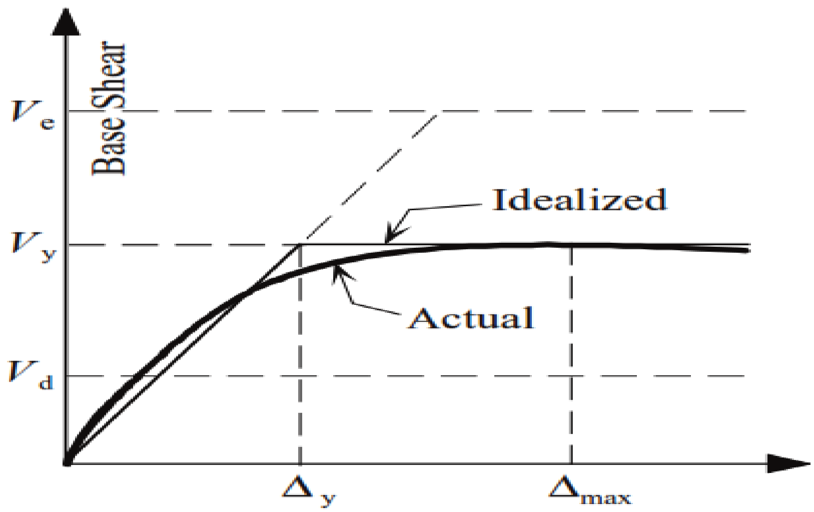


Figure 2.3: Typical global structural response of a building structure (Elnashai and Sarno, 2008).

Thus the ductility factor is defined as

$$\mu = \frac{\Delta_{max}}{\Delta_y} \tag{2.3}$$

the ductility reduction factor as

$$R_{\mu} = \frac{V_e}{V_{inel}} \tag{2.4}$$

and the force reduction factor as

$$R = \frac{V_e}{V_d} \tag{2.5}$$

Meanwhile ductility related force reduction factor, R defined through the ratio of the ground motion intensity, [Sa(T_1)/g] γ, where Sa(T_1) is the 5% damped spectral acceleration at the fundamental period of the structure (without P-delta effect). The γ represent the base shear coefficient, i.e., $\gamma = V_y / W$ where V_y being the yield base shear (without P-delta effects) as employed by Medina and Krawinkler (2003) and Zarien and Krawinkler (2009).

$$R_{?} = \frac{S_a(T_1)/g}{\gamma} \tag{2.6}$$

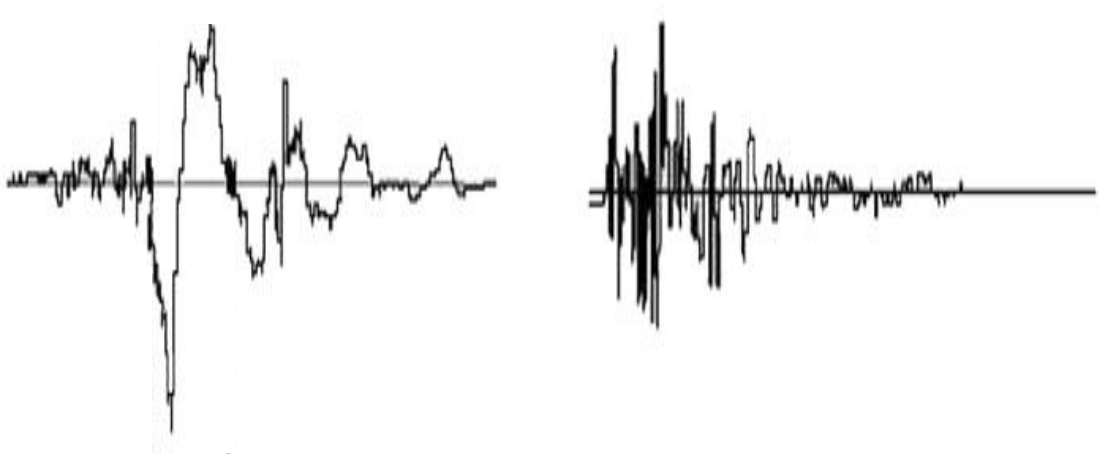
2.5 Ground motion

Ground motion acceleration contains different frequency, amplitude and duration, which reflect the earthquake source mechanism and site condition. Basically, there are three types of ground motions consisting of far field, near field (forward directivity) and near field (fling). Forward directivity occurs where the fault rupture propagates with a velocity close to the shear-wave velocity. Displacement associated with such a shear-wave velocity is largest in the fault-normal direction for strike-slip faults. Meanwhile, fling occurs in the direction of fault slip and therefore is not strongly coupled with the forward directivity It arises in strike-slip faults in the strike parallel direction as in the Kocaeli and Duzce earthquakes (Kalkan and Kunnath, 2006).

The far field earthquake (FFE) seems significantly different as compared to the near field earthquake (NFE) in term of velocity pulse. According to Bayraktar et al., (2009), NFE are characterized by a ground motion with the large velocity pulse. Not like

NFE, the FFE produces low input energy on the structure in the beginning of the earthquake.

This is because the FFE are recorded within a few kilometres of the rupture plane. Another distinguish factor is the distance between a structure to the epicentre of the earthquake. In the case of near-fault ground motions, the epicentre is within 20 km from the ruptured fault (Bray and Rodriguez-Marek, 2004). Meanwhile, for FFE, the distance to the epicentre of the earthquake is within 80 km (Razak, 2010). Figure 2.4 shows the comparison of the FFE and NFE in terms of velocities.



(a) NFE (b) FFE

Figure 2.4: Comparison of the (a) FFE and (b) NFE (Bayraktar et al., 2009).

Chopra et al., 2001 noted that the velocity-sensitive region for NFE motions is much narrower, and the acceleration-sensitive and displacement-sensitive regions are much wider, compared to FFE; the narrower velocity sensitive region is shifted to longer periods. Besides that, FFE motions imposed a smaller strength demand than NFE although for the same ductility factor.

2.6 Multi Degree of Freedom (MDOF) Systems

Most structures need to be modelled as Multi Degree of Freedom (MDOF) systems and illustrate a more complicated behaviour than Single Degree of Freedom (SDOF) systems, particularly in nonlinear ranges (Moghaddam and Mohammadi, 2001). Ruiz- Garcia and Miranda (2005) noted that the investigations focused in the evaluation of residual deformation demands of MDOF systems that represent frame building are limited. Many analysis were done using the SDOF system because SDOF systems have just one mode and they are less complicated then MDOF system and the investigation of the MDOF systems with higher modes and nonlinear behaviour is necessary (Vaseghi Amiri et al., 2008).

However MDOF system seismic behaviour can be approximated with certain accuracy by equivalent SDOF systems whose properties are computed by conducting pushover analyses (Naeim, 2001). Themelis (2008) suggest a simpler option to assess the performance of structures is pushover analysis or simplified nonlinear static analysis, even though this also requires as much as possible detailed mathematical models of MDOF systems.

For dynamic time history analysis MDOF systems required as many detailed mathematical models. For example, structures, together with information on ground motion characteristics, render it quite impractical for everyday use, especially when overly complex structures is required to be considered.

According to Themelis (2008), this method assumes that the response of a structure can be predicted by the first, or the first few modes of vibration, which remain constant throughout its response time. It involves the incremental application of loading that follows some predetermined pattern, until the failure modes of the structure can be identified, thus producing a force-displacement relationship or capacity curve, which gives a clear indication of the nonlinear response. The resulting displacement demands from the preceding analysis are then checked and the structural performance of the elements is assessed.

2.7 Method of Analysis

There are many methods in assessment of building performance during earthquake. The following methods are usually used in the assessment of the building performance according to Eurocode 8:

1) Static analysis (commonly known as “pushover” analysis), using equivalent seismic forces obtained from response spectra for horizontal earthquake motions.
2) Dynamic (time-history or response-history) analysis, either modal response spectrum analysis or time history analysis with numerical integration using earthquake records.

2.7.1 Nonlinear static analysis

For the majority of buildings, equivalent static analysis procedures can be used although earthquake forces are of dynamic nature.These methods generally determine the shear acting due to an earthquake as equivalent static base shear. It depends on the weight of the structure, the dynamic characteristics of the building as expressed in the form of natural period or natural frequency, the seismic risk zone, type of structure, and geology of the site and importance of the building.

Pushover analysis is used to quantify the resistance of the structure to lateral deformation and widely accepted as a rapid and reasonably accurate method (Chandrasekaran, 2009). There is a great saving in time when performing the pushover analysis as compared with the full nonlinear dynamic analysis. In seismic design and evaluation of structures, pushover analyses are commonly used as indicator of structural yielding and potential failure mechanisms.

In general, a sequence of inelastic static analysis is performed on the structural model of the building by applying a predefined lateral load pattern which is distributed along the building height. The lateral forces are then monotonically increased until it becomes unstable and reaches the collapse state (force controlled) or its roof displacement reaches the predetermined limit (displacement controlled) (Ramamoorthy, 2006).

The pushover analysis become a useful tool for preliminary design and assessment because the proposed bounds for collapse loads obtained in closed form, which fit with pushover analysis to a good accuracy. The pushover technique allows tracing the sequence of yielding and failure of the member beside provides useful information on the overall characteristics of the structural system.

Results of pushover analysis demonstrate resistance of the building in terms of story shear force versus top displacement, commonly referred to as the capacity curve of the building as shown in Figure 2.5. In certain cases, the numerical studies conducted show that the design base shear computed using nonlinear static pushover, for an accepted level of damage like collapse prevention, predicts the response value closer to the upper bounds obtained by plasticity theorems.

Since the pushover analysis is approximate in nature and is based on static loading, as such it cannot represent dynamic phenomena with a large degree of accuracy. It may not detect some important deformation modes that occur in a structure subjected to severe earthquakes, and it may significantly from predictions based on invariant or adaptive static load patterns, particularly if higher mode effects become important.

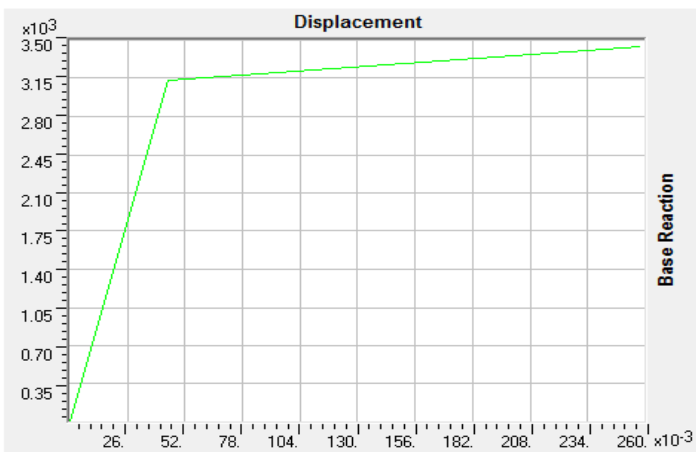


Figure 2.5: Capacity Curve (SAP 2000)

From the pushover analysis, the performance level of the building can be determined. The performance level of the building depends on the formation of plastic hinges of the members. FEMA 273 (1997) define force-deformation criteria for hinges used in pushover analysis.

As shown in Figure 2.6, five points labelled A, B, C, D, and E are used to define the force deflection behaviour of the hinge and three points labelled Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) are used to define the acceptance criteria for the hinge (FEMA 356, 2000). The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in the FEMA 273 (1997).

In Figure 2.6, the yield point of strength and deformation was presented by point B, whereas Point C represents the ultimate points. Point D reflects the strength degradation of the member capacity, whereas Point E represents total failure of the member. Value used for SAP2000 is the Point B-C-D-E values normalized to yield value of strength and deformation.

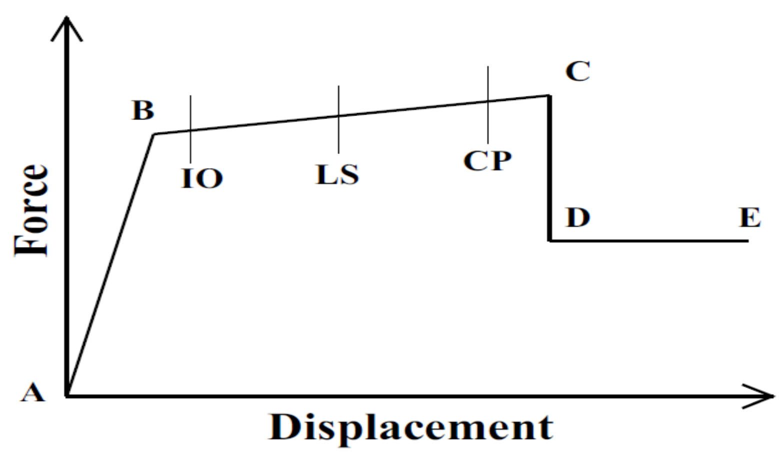


Figure 2.6: Strength and deformation points (FEMA 356, 2000).

In the SAP 2000 programs, the sequence of yielding and failure of the member are illustrated in the two dimensional. Figure 2.7 show the sequence of the hinges from yielding until the member failure. The colour's dot represented the critical response of the structure. The colourful dots relates to the points B to E as shown in the Figure 2.6. For example, the point B represents by the pink colour dot, dark blue dot represents IO and light blue dot represents LS point and so on. From the figure, the top members will fail first followed by the bottom members.

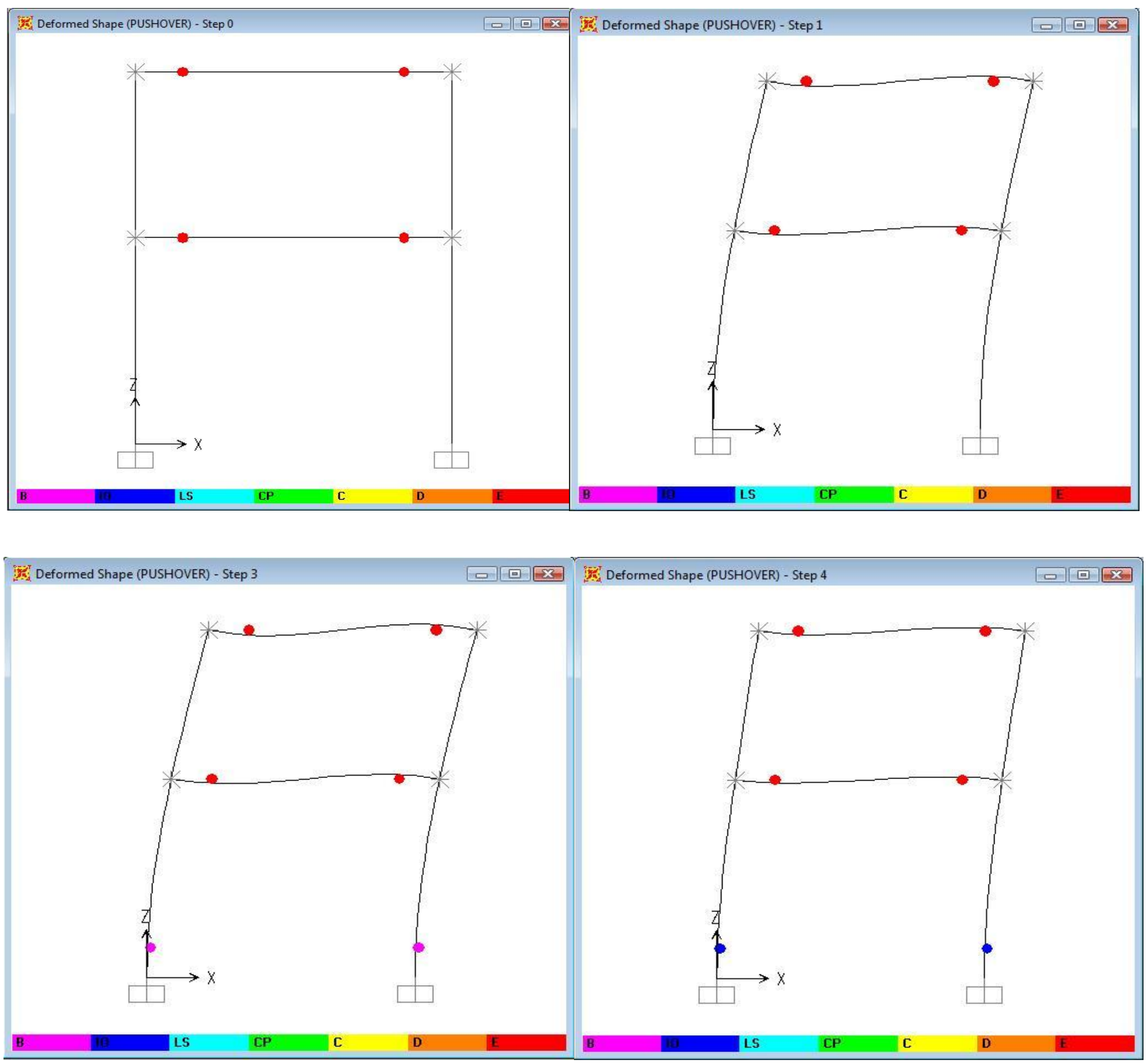


Figure 2.7: Sequences of hinges formation (SAP 2000).