SEISMIC PERFORMANCE OF REINFORCED CONCRETE SCHOOL IN SABAH

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ABSTRAK

Salah satu kaedah untuk menilai prestasi seismik adalah analisis ‘time history’ tidak lelurus. Ini kerana model struktur mengalami gempa bumi sebenar melalui rekod pergerakan tanah. Amalan semasa dalam kejuruteraan gempa bumi hanya mempertimbangkan gempa bumi tunggal dalam pemodelan dan analisis. Pada hakikatnya, gegaran gempa bumi akan berlaku berulang kali. Ini dikenali sebagai gempa bersiri. Gempa bumi di Ranau yang berlaku pada 5 Jun 2015 juga tidak terjadi secara tunggal, tetapi diikuti oleh beberapa siri gempa susulan. Projek ini adalah untuk menyiasat prestasi konkrit bertetulang bangunan sekolah di Sabah apabila dikenakan gempa bumi bersiri. Sebanyak tiga model sekolah yang mempunyai 2, 3 dan 4 tingkat telah digunakan untuk projek ini. Semua model telah direka berdasarkan BS8110 untuk mewakili bangunan sekolah konkrit bertetulang sedia ada. Sebanyak tujuh pergerakan tanah untuk kedua-dua gempa bumi tunggal dan gempa bumi bersiri telah dipertimbangkan dalam analisis ‘time history’ tidak linear ke atas semua model dengan menggunakan program SAP2000. Semua model diandaikan dibina di kawasan seismik sederhana di Sabah, Malaysia. Objektif kajian ini adalah untuk mengkaji prestasi bangunan sekolah konkrit bertetulang di Sabah apabila dikenakan gempa bumi bersiri. Berdasarkan beberapa siri analisis ‘time history’ tidak lelurus, kajian ini mendapat kesimpulan bahawa kejadian gempa bumi bersiri telah menyumbang anjakan sisi maksimum sebanyak 58% - 73% lebih tinggi berbanding gempa bumi tunggal. Selain itu, kajian ini mendapati kejadian gempa bumi bersiri telah menyumbang nisbah anjakan antara tingkat sebanyak 60% - 67% lebih tinggi berbanding gempa bumi tunggal. Oleh itu, gempa bumi bersiri perlu diambil kira untuk merekabentuk bangunan baru serta penilaian untuk tujuan penyelenggaraan dan pemuliharaan bangunan yang sedia ada.

ABSTRACT

One of the methods to evaluate the seismic performance is the nonlinear time history analysis. This is because the structural models are imposed to the real earthquake ground motion records. Current practice only considers single earthquake in the analysis. In reality, the earthquake tremors will occur repeatedly. This is known as multiple earthquakes. Ranau earthquake that occurred on 5 June 2015 also was not a single earthquake, but followed by several number of aftershock. This project is to investigate the performance of reinforced concrete (RC) school buildings in Sabah when subjected to multiple earthquakes. A total number of three models of school which have 2, 3 and 4 storeys has been used for the project. All models have been designed based on BS8110 to represent the existing RC school buildings. A total of seven ground motion for both single earthquake and multiple earthquakes have been considered in nonlinear time history analysis on all models by using SAP2000 program. All models are assumed to be located in moderate seismic region in Sabah, Malaysia. Objective of this work is to study the performance of RC school buildings in Sabah when subjected to multiple earthquakes. Based on a series of nonlinear time history analysis, this study concludes that the action of multiple earthquakes has contribute around 58% - 73% higher maximum lateral displacement compared to the single earthquake. This study also concludes that multiple earthquake significantly contributes around 60% - 67% higher interstorey drift ratio compared to the single earthquake. Thus, multiple earthquakes consideration should be taken in order to design new buildings as well as the evaluation for maintenance and rehabilitation of existing buildings.

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LIST OF SYMBOLS

Δ	Displacement
Δ_{max}	Maximum displacement
R	Force Reduction Factor
V	Shear
Q_k	Live load
G_k	Dead load
f_{cu}	Concrete compressive strength
f_y	Yield strength of steel
C_t	Coefficient
F_b	Base shear force
q	Behaviour factor
γ	Base Shear Coefficient
T1	Fundamental Period
T_B	Lower limit of the period of the constant spectral acceleration branch Beginning of the constant displacement response
T_D	range of the spectrum
S	soil factor
$S_d(T)$	design spectrum
B	lower bound factor for the horizontal design spectrum (0.2)

LIST OF ABBREVIATIONS

FEMA	Federal Emergency Management Agency
BS	British Standard
MMD	Malaysia Metrology Department
JMGM	Jabatan Mineral and Geoscience Malaysia
RC	Reinforced Concrete
FFE	Far Field Earthquake
MDOF	Multi Degree of Freedom
NEHPR	National Earthquake Hazards Reduction Program
NFE	Near Field Earthquake
IDR	Interstorey Drift Ratio
SDOF	Single Degree of Freedom
UBC	Uniform Building Code
USGS	U.S. Geological Survey
DNFS	Near Field Single Earthquake on Soil Type D
DNFR	Near Field Multiple Earthquake on Soil Type D

CHAPTER 1

INTRODUCTION

1.1 Background

Malaysia in one of the country located at the equator of the globe situated far away from the active seismic fault zone. The nation is occasionally considered as safe from earthquake disaster as the nation is located outside the earthquake region. Therefore, the construction practice in Malaysia does not have awareness upon the earthquake disaster regarding to the building's design precaution. Generally, existing reinforced concrete (RC) buildings in Malaysia were designed by using BS8110 (1997) without any earthquake safety precaution. After Ranau earthquake on June 2015, Malaysia citizen with different hierarchy, from government leader, local authorities, researchers, engineers and public citizen keep asking the same question on how safe the buildings when subjected to earthquake loading if the earthquake happened again especially in Sabah, where there is no provision for seismic code yet. It is always being a fear to people to facing an earthquake since the magnitude and vibration are unpredictable.

The movement of tectonic plate deep underneath the earth has caused many vibrations onto the earth surface. Some of them has bring a lot of disaster to the earth including man-made structures like bridges, buildings, slopes and roads, which caused the loss of properties, injuries and fatality as well as the changes of landform due to landslides and tsunami. It has been recorded all over the world that earthquake disaster has brought many damages on earth. On 5th June 2015, a moderate tremor struck the northwest of Ranau, Sabah around 7:15am local time. Malaysian Metrology Department (MMD) has reported that the recorded magnitude of the tremor is 5.9 Richter scale which lasted for 30 second. The epicenter was located 16 km northwest from Ranau with the depth of 54 km beneath the earth. The tremors were felt in Ranau, Kundasang, Tambunan, Pedalaman, Kota Kinabalu and Kota Belud (Adiyanto, 2016). Although the magnitude of the earthquake is considered as moderate, but it has brought a lot of impacts to the local people.

During the tragedy, plenty of buildings has damaged and distorting their economy resources such as plantation. Some of them are facing trauma and fears to stay inside their house especially at the multi-storey houses. Figure 1.1 presents the epicentre of 2015 Ranau earthquake.

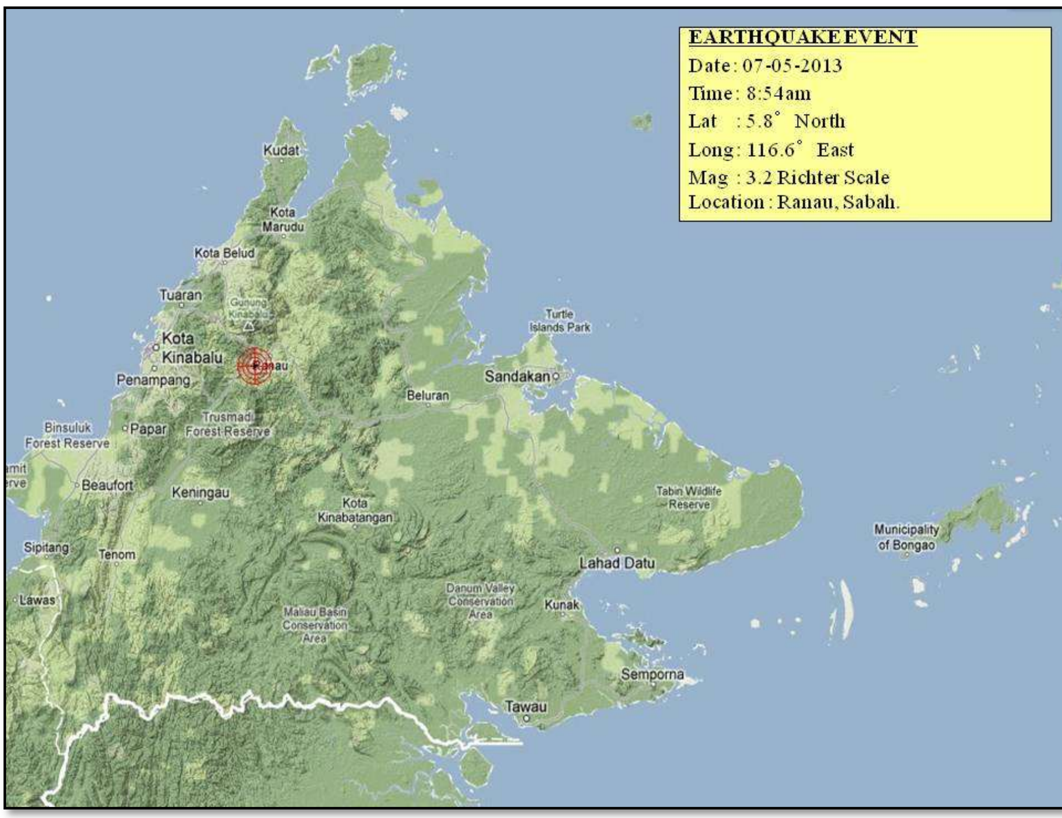


Figure 1.1 Epicentre of Ranau, 2015 earthquake (MMD,2015)

Earthquake does not kill people, but the disaster caused by the earthquake will. To prevent injuries and fatality caused by earthquake strike, every structural engineer should take into account all the consideration to ensure that the man-made structures are able to withstand seismic action. Thus the design for future structures should be designed according to Eurocode 8 (2004) as well as the inspection and assessment of existing buildings whether the buildings are able to survive under seismic action.

In earthquake engineering, the interstorey drift ratio is very important to be investigated. This parameter can be used as early description for engineers to predict how safe the buildings during real earthquake event. High interstorey drift ratio will cause damage to the non-structural elements such as window frames, ceiling, electrical wiring, etc. Then, the damage will occur to the structural element, i.e. beams and columns, before the whole structure might experience collapse. Therefore, this study is significant for Malaysia especially Sabah scenario.

1.2 Problem Statement

A study related to seismotectonic setting of Malaysia conducted by Jabatan Mineral and Geoscience Malaysia (JMGM) has found that Malaysia is considered as a country with low seismicity except for Sabah state(MOSTI,2009). The Ranau earthquake on 5 June 2015 with a magnitude of M_w 5.9 has recorded as the strongest tremors to affect Malaysia for the last 45 years. This tragedy has given a very big challenge to the nation especially in construction industry to come out with new consideration for analysis and design of structure against seismic action.

Occasionally, analysis and design for seismic resistance structure by using Eurocode 8 (2004) and FEMA 356 (2000) only considering single earthquake for every single structure. Nevertheless, the nature of earthquake events shows that most of the earthquake events were occurred repeatedly. It is always seen that there will be certain number of tremors after the first one. This is called as multiple earthquake (Hatzigeorgiou and Beskos, 2009). The 2015 Ranau earthquake tragedy also demonstrated that there are more than hundreds tremors occurred after the first tremor. Table 1.1 shows the list of selected tremors during the 2015 Ranau earthquake.

Very few studies have been reported in the literature regarding to the multiple 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 Liolos (2010) noted that the sequences of ground motion have a significant effect on the response and hence, on the design of the RC buildings.

Table 1.1: List of selected tremors during the 2015 Ranau earthquake. (MMD,2015)

Date	Time	Location	Latitude	Longitude	Magnitude
5/6/2015	7.15am	Ranau	6.1◦U	116.6◦T	6.0
5/6/2015	7.28am	Ranau	6.1◦U	116.6◦T	3.6
5/6/2015	9.51am	Ranau	6.2◦U	116.5◦T	3.9
5/6/2015	12.05pm	Ranau	6.1◦U	116.5◦T	4.0
5/6/2015	2.40pm	Ranau	6.1◦U	116.6◦T	3.0
5/6/2015	1.45pm	Ranau	6.1◦U	116.6◦T	4.5
5/6/2015	6.58pm	Ranau	6.1◦U	116.6◦T	3.2
5/6/2015	10.36pm	Ranau	6.1◦U	116.6◦T	3.3
6/6/2015	1.32am	Ranau	6.1◦U	116.6◦T	3.7
6/6/2015	2.19pm	Ranau	6.1◦U	116.6◦T	3.4

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

The damage of the structures is directly related to the interstorey drift ratio of every storey of the building (Hatzigeorgiou, 2010b). Thus, evaluation on their relationship is very important for structural performance. Interstorey drift ratio of multiple earthquakes is relatively higher compared to single earthquake (Hatzigeorgiou and Liolos, 2010). Figure 1.2 shows the effect of sequences of the ground motion. Therefore, it is important to study the vulnerability of existing RC buildings in Malaysia when subjected to multiple earthquakes.

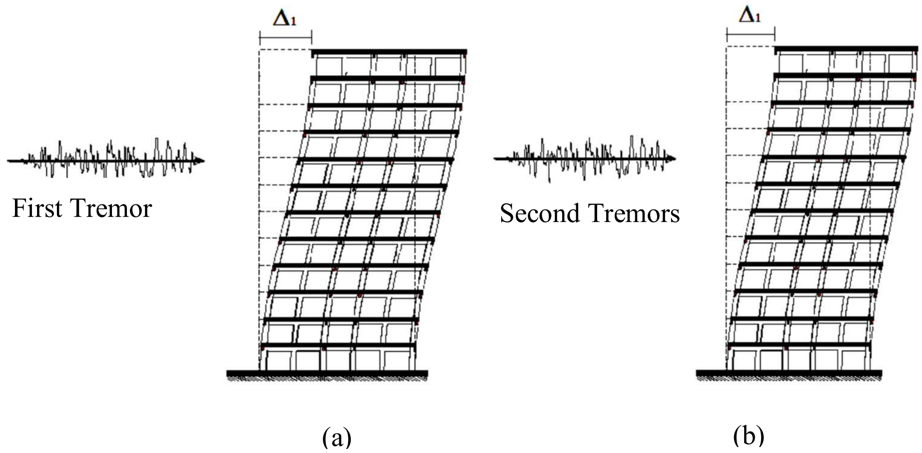


Figure 1.2 Effect of sequence of ground motion; (a) First tremor, (b) Second tremor (Hatzigeorgiou, 2010a)

1.3 Objectives

Objectives of the study are:

i. To investigate the maximum lateral displacement of RC school building in Sabah Malaysia when subjected to single and multiple earthquakes.
ii. To study the seismic performance of RC school building in Sabah, Malaysia in terms of interstorey drift ratio (IDR) when subjected to single and multiple earthquakes

1.4 Scope of Work

This study covered and focused in the following aspect:

i. Three model of RC school buildings which have 2, 3 and 4 storey respectively designed based on BS8110 (1997) by using ESTEEM software.
ii. Section properties of the designed structures by using CUMBIA (Montejo and Kowalsky, 2007) to determine the nonlinear properties of the structural elements.
iii. Scaling of 7 single NFE ground motion by using seismosignal and Matlab to match the designed response spectrum of Sabah according to Eurocode 8 (2004).
iv. The reference peak ground motion a_{gR}=0.12g has been used for scaling of ground motion to represent the seismicity of Sabah based on the Malaysia Earthquake Hazard Map (MOSTI,2009)
v. 7 numbers of single ground motion and 7 random combination of single earthquake to represent multiple earthquakes has been considered in nonlinear time history analysis on all models.
    a) Case 1: single ground motion (mainshock).
    b) Case 2: multiple ground motion (foreshock-mainshock-aftershock).
vi. All the models are assuming to be built on soil type D
vii. The seismic performance of the RC school buildings had been evaluated based on the lateral displacement and IDR of every single storey.

1.5 Thesis Outline

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

Chapter 2 covers all aspect discussed in this study together with literature review. This chapter is discussing about single earthquake, multiple earthquake phenomenon and the analysis used in the study.

Chapter 3 further discusses on the model and the ground motion sequence used to analyse the model. In this chapter also briefly explain step-by-step procedures.

Chapter 4 is discussing the result of the study. This chapter covers the effect of single and multiple near field earthquake (NFE) on the lateral displacement of every storeys. Besides, this chapter also covers the discussion about the effect of single and multiple earthquake towards the lateral displacement and IDR of every models.

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

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Structural buildings often undergo displacement when subjected to lateral load (seismic action). The building will be shifted rapidly from its original position due to the seismic force that creates sudden force towards the building. This phenomenon is also influenced by the ground motion due to the earthquake event. In fact, the seismic action that generated by the earthquake event can cause significant damage towards a structure within short distance of a fault. The studies in earthquake engineering are often subjected towards the effects of high frequency components of an earthquake onto the buildings.

In this chapter, some of the terminology will be highlighted for better understanding in this study such single and multiple earthquakes, ground motion, and interstorey drift ratio (IDR).

2.2 Multiple earthquake phenomenon

Current study regarding to the analysis of structural performance only consider single earthquake. Nevertheless the nature of the earthquake, its may occur multiplely. This is technically called as multiple earthquakes (Hatzigeorgiou and Beskos,2009). The earthquake may occur multiplely and it is difficult to predict the frequency of the earthquake hits the ground (Ellen, 2000). This phenomenon is very dangerous to the structures in terms of performance. Current study regarding to the multiple ground motion has ignored its effect toward the buildings in the code.

Basically, after the ground shaking, the first wave of the seismic action will hit the buildings and caused some displacement of the buildings to occur. After the first hit, the upcoming seismic force produce by the ground motion causing a permanent displacement that are obviously cumulated and give a maximum displacement appears to be increased as a result (Hatzigeorgiou, 2010a).

It can be seen that the structure are already damaged during the first hit. The not yet repaired structures may become completely not able to sustain the seismic load effect until the end of the earthquake sequence. Amadio et al, 2003 has considered that the accumulation of damage are depends on the type of hysteric structural behaviour and on the characteristics of the seismic events.

2.2.1 Foreshock, Mainshock and Aftershock

Multiple earthquakes phenomenon consists of sequences which are known as foreshock, mainshock and aftershock. Mainshock is the one with largest quake occuring in between any foreshcock and aftershock. Foreshocks are the smaller magnitude earthquakes that come before the bigger quake and not all mainshock have foreshock.

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

In this figure, mainshock have larger magnitude compared to foreshock and aftershock. For example, in the Northidge earthquake the mainshock which is the largest, had magnitude of 6.7. There were no foreshock, but immediately after the mainshock 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). Aftershock usually have an orderly and steady rate of decay which means that they becomes less frequent with time. This does not mean that aftershock necessarily decrease in magnitude with time.

Table 2.1 The detailed of three earthquake (usgs, 2011)

Time, PDT	Magnitude	Latitude	Longitude	Depth	Designatio n
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

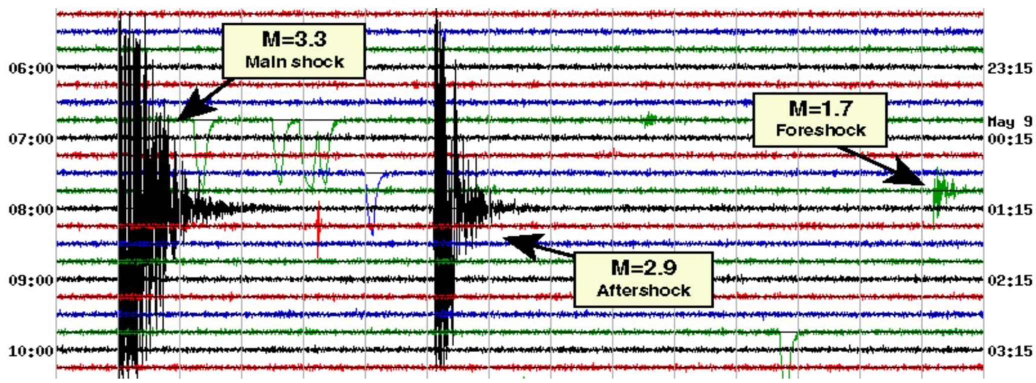


Figure 2.1 Comparison between foreshock, main shock and aftershock (USGS,2011)

Time is an important factor affects the aftershock since there might be many numbers of aftershock after the mainshock. Aftershock decrease proportionately to the time since the mainshock happened and the bigger earthquake have larger aftershock. The event might be continuously happening within the first hour or maybe a day, week, month even a year of the earthquake event.

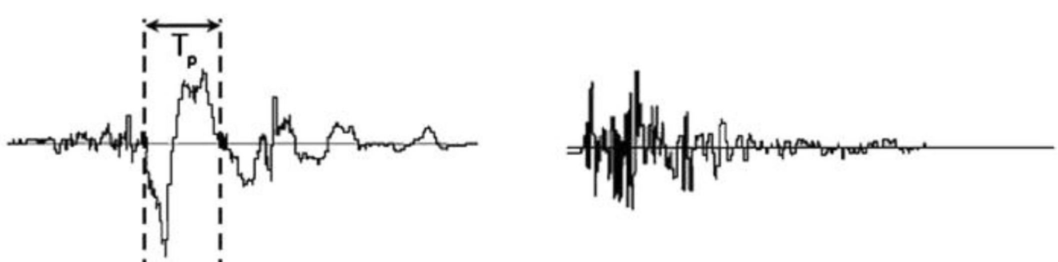
On average, the bigger mainshock will have followed by the bigger aftershock. Difference of magnitude between the mainshock and largest aftershock ranges from 0.1 to 3 or more, but the averages is 1.2 (USGS, 2011). In fact, there are more small scale aftershocks than the largest one and the decay can be noticed more quickly since the largest aftershock are already less frequent. Larger aftershock can occur months or even a few years after the main shock.

2.3 Ground Motion

Ground motion is a movement of the earth's surface resulted from earthquake or explosion. Ground motion produced by wave that was generated from sudden slip on a fault or sudden pressure at the explosive source and travel through the earth and along its surface. Ground motion acceleration contain different frequency, amplitude and duration that affected by the source of mechanism and condition of the site. There are three types of ground motion which known as 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 formal directivity. It arises in strike-slip fault in the strike parallel direction as in the Kocaeli and Duzce earthquake (Kalkan and Kumnath, 2006)

The different of the far field earthquake (FFE) and near field earthquake (NFE) is only the velocity pulse. According to Bayraktar et al, (2013), NFE are characterized by a ground motion with the large velocity pulse. Significantly different to NFE, the FFE generates low input energy on the structure in the beginning of the earthquake. The reason is because the FFE are recorded within a few kilometers from the rupture plane.

Furthermore, it can be seen that another distinguish factor is the distance between the structure to the epicentre of the earthquake. For near-fault ground motion, the epicentre is within 20 km from the ruptured fault (Bray and Rodriguez-Marek, 2014). Meanwhile for FFE, the distance to the epicentre of the earthquake is within 80 km (Razak, 2010). Figure 2.2 shows the comparison of FFE and NFE in terms of velocities.



(a) NFE (b) FFE

Figure 2.2 Comparison of the (a) NFE and (b) FFE (Bayraktar et al, 2009)

Razak, 2010 noted that the velocity-sensitive region for NFE motion 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 motion imposed a smaller strength demand than NFE although for the same ductile factor.

2.4 Method of Analysis

Earthquake engineering has become one of the most important fields in engineering world. There are many methods has been introduced in assessment of building performance during earthquake events. The following approaches are oftenly used in the assessment of the building performance according to Eurocode 8 (2004):

1) Static analysis (technically known as "pushover" analysis), are conducted under the factored gravity load combination and static lateral earthquake forces. Pushover analysis is carried out under permanent vertical loads and gradually increasing lateral load to calculate the deformation as well as damage pattern of structure.
2) Dynamic (time-history or response-history) analysis, either modal response spectrum analysis or time history analysis with numerical integration using earthquake records.

2.4.1 Nonlinear static analysis

These methods are generally determining the shear force acting due to an earthquake as equivalent static base shear. Generally, it depends on the self-weight of the structure. Basically, equivalent static analysis procedure can be used for majority of buildings, although earthquake forces are of dynamic nature. The dynamic characteristics of the buildings as expressed in the form of natural period or natural frequency, the seismic risk zone, type of structure, geology of the site and the importance of the buildings as well.

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). Pushover analysis requires less period of time to be conducted as compared to the full nonlinear dynamic analysis. It is also commonly being used as indicator of structural yielding and potential failure mechanism in seismic design and evaluation of structures.

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

Thus, the pushover analysis has become a useful tool for preliminary design and assessment due to 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.

The result of pushover analysis represents and demonstrates resistance of the buildings in term of story shear force against top displacement. It is generally referred as the capacity curve of the buildings as shown in Figure 2.3. Nevertheless, in certain cases, the numerical studies conducted shows 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.

Besides, the pushover analysis is not able to represent dynamic phenomena with a large degree of accuracy since it is approximately in nature and it is only based on static loading. It might not able to detect some important deformation modes that occur in a structure subjected to severe earthquakes. Furthermore, it might significantly differ from prediction based on invariant or adaptive static load patterns, particularly if higher mode effects become important.

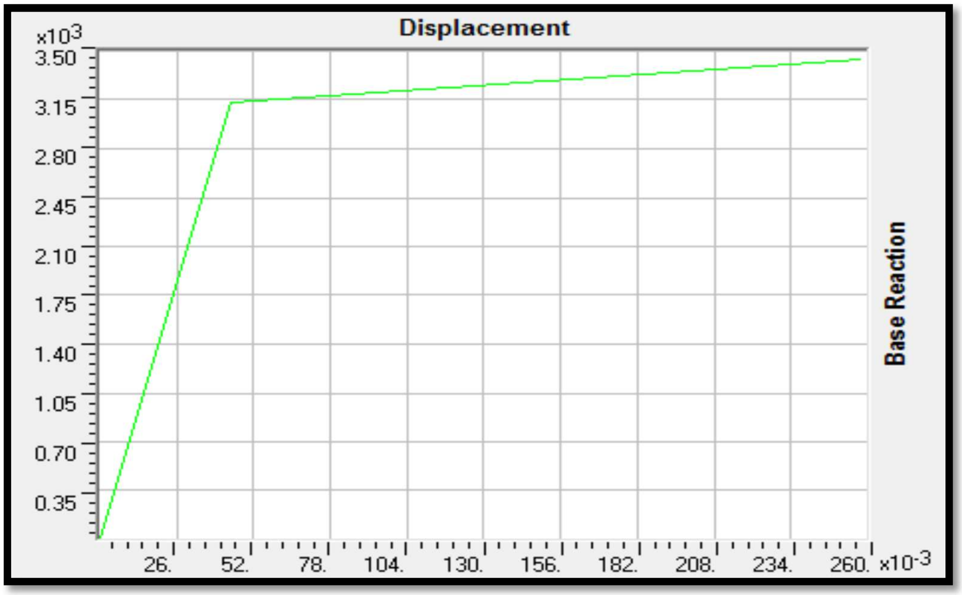


Figure 2.3 Capacity Curve (SAP 2000)

As a result, the performance level of the building can be determined from the pushover analysis. It depends on the formation of plastic hinges of the members. FEMA 273 (1997) stated that force-deformation criteria for hinges used in pushover analysis. Figure 2.4 shows five labeled A, B, C, D, and E are used to define the force deflection behavior of the hinge and three points labeled immediate occupancy (IO), life safety (LS) and collapse prevention (CP) are used to define the acceptance criteria for the hinge (FEMA 356, 2000). Varies value are assigned to each of the points depending on the type of member as well as many other parameters defined in the FEMA 273 (1997).

From Figure 2.4, point B representing the yield point of strength and deformation whereas the ultimate point was represented by the point C. Point D reflects the strength degradation of the member capacity and point E represent the total failure of the members. Value used for SAP2000 is the point B-C-D-E values normalized to yield value of strength and deformation.

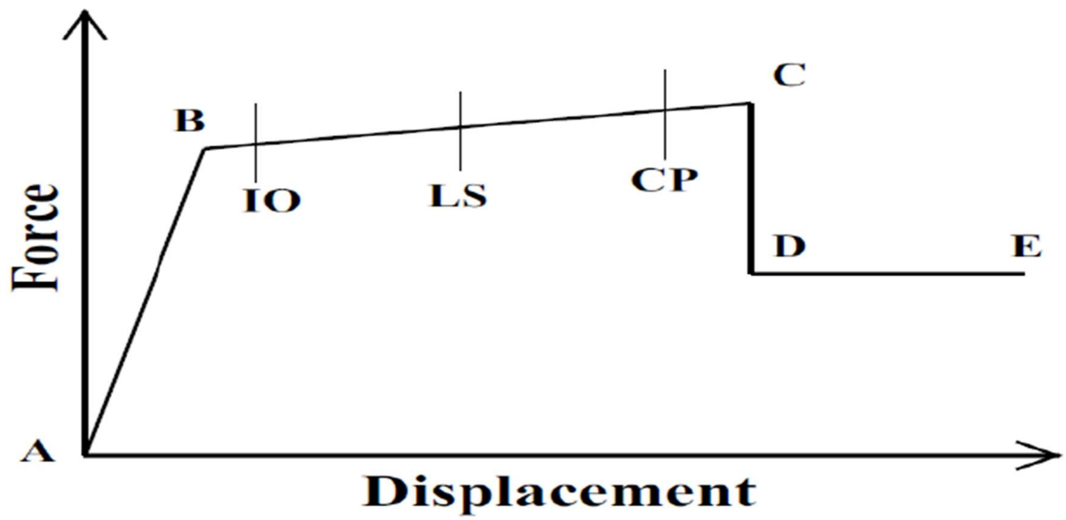


Figure 2.4 Strength and deformation points (FEMA 356, 2000)

In the SAP 2000 programs, the sequence of yielding and failure of the member are illustrated in the two dimensional. Figure 2.5 describe the sequence of the hinge from yielding until the member failure. The critical response of the structure is represented by the coloured dot. The colourful dots relates to the point B to E as shown in Figure 2.4. For example, the point B represents by the pink colour dot, dark blue dot represent IO and light blue dot represent LS point and so on. From the figure, the top member will fail first followed by the bottom members.

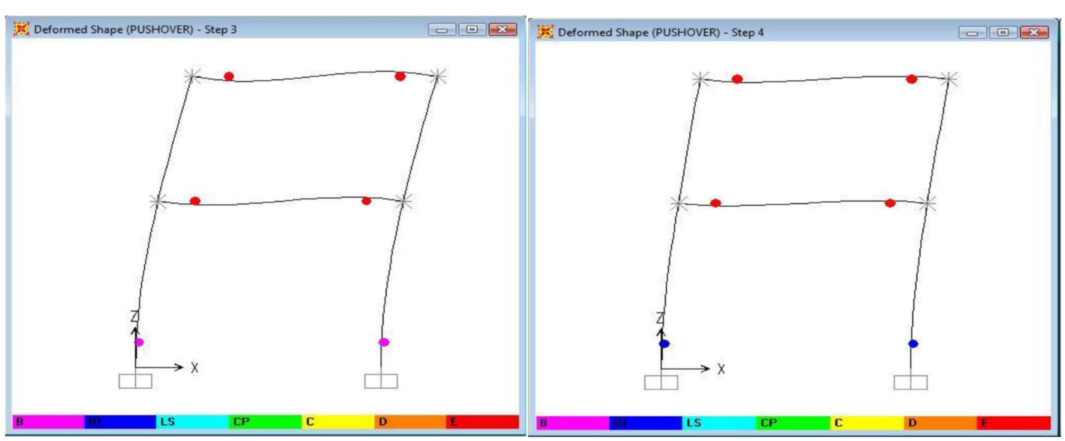


Figure 2.5 Sequence of hinges formation (SAP 2000)

2.4.1 Nonlinear Dynamic (time history) Analysis

The dynamic time-history analysis can be classified as either linear elastic or inelastic analysis (Chopra, 1995). This method simulates the response of structural system when subjected to earthquake ground motion records, either real of artificial. Several commercial packages software for the 3D elastic analysis of structures is available and is in widespread use such as SAP2000, ETABS, and SPACE GASS

2.5 Collapse Structural Analysis

After the buildings subjected to an earthquake events, the buildings may be experiencing partial or total collapse. Current studies in earthquake engineering have enable researcher to stimulate and calculate the structures' risk of collapse under seismic loading, providing explicit measures of collapse risk which can be used to compare the risks posed by different types of structures.

Collapse defined as incapacity of a structural system, or a part of it, to maintain gravity load-carrying capacity under seismic excitation (Ibarra and Krawinkler, 2005). Collapse of a structure normally occurs if vertical load-carrying components fail in compression or if shear transfer is lost between horizontal and vertical components (e.g shear failure between a flat slab and a column). There are several factor can cause global collapse to occurs. As an example the spread of an initial local failure from element to element may result in cascading or progressive failure.

2.6 Effect of Multiple Earthquake on Structural Performance

This section discusses the review on previous researches related to effect of multiple earthquake on structural performance. As mention in Chapter 1, current scenario all over the world has shown that earthquakes always occur multiply. Especially in a big earthquake event, there is no single tremor but always followed by several tremors. It can be said that the earthquake always started by foreshock, then followed by the mainshock before the aftershock event (Ruiz-Garcia, 2014)

Among the earliest work to study the effect of multiple earthquake on structural performance was conducted by Amadio et al. (2003). Single Degree of Freedom (SDOF) model was used in dynamic analysis considering one real and two artificial multiple earthquake records. In artificial earthquakes, a gap of 100 seconds was assigned between two consecutive events. This is important for the SDOF model to cease the vibration caused by the first event. From their work, the authors concluded that for long period structure ($T_1 > 2.0$ sec) the response due to multiple earthquake is very similar to the response of the same structure under single earthquake excitation. However, the multiple earthquake generally requires an increase in strength with respect to the single earthquake event especially for low period structures ($T_1 = 0.1$ sec to $T_1 = 1.5$ sec). Therefore, the assessment of vulnerability and seismic risk for such structures should consider the multiple earthquake phenomenon.

After several years, which is on 2009, Hatzigeorgiou and Beskos has conducted dynamic analyses on SDOF models with various period of vibration ($T_1 = 0.1$ sec to $T_1 = 3.0$ sec). They have been used Four type of different soil classes namely as A, B, C, and D represent the hard rock, soft rock, stiff soil, and soft soil, respectively. Between two consecutive events, a time gap equal to 3 times of duration of single event had been applied to cease the vibration of the SDOF models. Based on their analyses, the authors had concluded that the traditional seismic design procedure which is based on the isolated design earthquake (single earthquake) should be reconsidered.

Hatzigeorgiou (2010a) has develop ductility demand control under multiple NFE. The author has used two series of combination in order to generate the artificial multiple earthquake. For the first series, the author suggested that for every seismic event with Peak Ground Acceleration (PGA) equal to $A_{g,max}$ there will be three other events with PGA equal to $0.7767A_{g,max}$. This means that for every mainshock with magnitude equal to $A_{g,max}$, there will be three other events either foreshock and/or aftershock with magnitude equal to $0.7767A_{g,max.}$

Furthermore, in second series, Hatzigeorgiou (2010a) suggested that for every seismic in the second series, Hatzigeorgiou (2010a) suggested that for every seismic event with PGA equal to $Ag.max$ there will be two other events with PGA equal to

0.8526*Ag.max*. This means that for every mainshock with magnitude equal to *Ag.max*, there will be two other events either foreshock and/or aftershock with magnitude equal to 0.8526*Ag.max*. Therefore, for artificial multiple earthquake the foreshock and aftershock records shall be scaled down based on the relevant coefficient as mentioned before. Based on inelastic time history analyses, the author concluded that the ductility demand induced by the multiple earthquake is higher than single earthquake. It may lead to greater damage even collapse. Hence, it is impractical to ignore the multiple earthquake phenomenon. The structural analysis cannot rely to the single earthquake only. However, the results between the first and second series of artificial multiple earthquake are not significantly different

As an expansion to the previous work, the ductility demand spectra for SDOF system considering multiple NFE and FFE had been proposed by Hatzigeorgiou (2010b). From inelastic time history analysis, it was found that the ductility demand required by NFE and FFE were different for both single and multiple earthquakes. The latter strongly affecting the ductility demand which lead to greater damage. Therefore, the influence of multiple earthquake should be taken into account in order to evaluate the ductility demand of a structure when subjected to earthquake. Finally, the author strongly suggested that the traditional seismic design procedure should be reconsidered.

The effect of multiple earthquake on maximum IDR was presented by Hatzigeorgiou and Liolios (2010). Inelastic analyses had been conducted on eight models considering 3 and 8 storey RC frames represent office buildings. The artificial multiple earthquake had been generated as input for that analysis. In order to cease vibration of structural models due to previous tremor, a time gap equal to 100 seconds between two consecutive seismic events had been recommended. This means that after being imposed by variable magnitude of dynamic load for a certain period, the structure will experience zero dynamic loads for 100 seconds. This duration is enough to cease the vibration of the structure before being imposed by other dynamic load representing next seismic event. The authors concluded that the displacement, as well as the IDR due to action of multiple earthquake was higher compared to the single earthquake. This leads to greater damage where the IDR might exceed the permissible limit.

The performance of steel frame buildings under multiple earthquake excitation had been investigated by Ruiz-Garcia and Negrete-Manriquez (2011). The nonlinear time history analysis had been conducted on the 4, 8, and 12 storey models considering 64 mainshock-aftershock ground motion records. The results concluded that the IDR are increased due to after-shock activities and it also depends on the storey height, *H* number of storey, N and fundamental period of vibration, T_1. It was also proven that the NFE caused higher magnitude of IDR compared to the FFE.

Ade Faisal (2012) conducted the nonlinear time history analysis on the 3, 6, 12, and 18 storey regular RC frames built on Soil Type B in Zone III in Greece. Each frame was designed based on behaviour factor, *q* equal to 1, 1.5, 2, 4, and 6. The author had concluded that the maximum storey ductility demand caused by multiple earthquake is about 1.3 to 1.4 times higher compared to the single earthquake. According to Ade Faisal et al. (2013) the maximum story ductility demand due to multiple earthquake for frames designed based on behaviour factor, $q \leq 2$ was small and negligible

Adiyanto and Majid (2014) conducted nonlinear time history analysis on three storey RC hospital buildings built on soil type D (soft soil) in Malaysia. The three storeys with two bays model was designed based on behavior factor, *q* equal to 2.3, 3.1, 3.9, 4.7, and 5.5. The author concludes that the action of multiple earthquake on frame tend to cause higher IDR compared to action of single earthquake. The authors concluded that, the action of multiple earthquakes on frames designed with behavior factor, $q \geq 3.9$ exceed the limits of 1.25% as recommended. So, lower value of behavior factor, *q* should be considered so that the buildings can survive the multiple earthquakes without structural damages.

Recently, Zhai et al. (2015) reported that the top displacement may increases up to 30% when subjected to multiple earthquake. The latter also caused greater structural damage than the single earthquake. Therefore, the authors strongly suggested that the multiple earthquake phenomenon cannot be neglected and the traditional response spectrum which is based on single earthquake should be further evaluated.

2.7 Summary

Multiple earthquakes were ignored in current seismic code of analysis and design of buildings. Thus, recent studies have been done on multiple earthquakes but mostly only focus on SDOF system. Therefore, it is important to extent SDOF to Multi Degree of Freedom (MDOF) systems for more realistic analysis. The famous analysis in an assessment is the pushover analysis and time history analysis. In addition, no previous studies conducted the nonlinear time history analysis considering single and multiple earthquake on RC school buildings in Malaysia. Therefore, this study covered that scope and the results are discussed in Chapter 4.

CHAPTER 3

METHODOLOGY

3.1 Introduction

This chapter explain the research procedure used in this study. The 2-dimensional (2D) generic school buildings models, the selection of ground motion and the analysis used in this study are discussed. The procedure has divided into three major phases consisting several steps in each phase. Figure 3.1 shows the flowchart that represents the overall procedure of this study.

The methodology started in phase 1 where 2D generic school buildings for 2 storey, 3 storey and 4 storey models located originally at Ranau, Sabah has been modelled. Then section analysis on the modelled models has been conducted by using CUMBIA set of coding to determine the nonlinear properties of the structural members of the models. Engineering assumptions such as the reinforcement capacity on every model is also discussed in detail. Furthermore, the modelling by using SAP2000 has been conducted on each model.

The second phase is to integrate the ground motion to be used for the nonlinear time history analysis. A total of seven numbers of near field earthquake (NFE) has been selected from all over the world. The ground motion is further randomly selected to generate the artificial multiple ground motion. Next, the single and multiple ground motion records are used on the SAP2000 program to perform the nonlinear time history analysis. Finally, the seismic performance of the models is evaluated based on the lateral displacement of each of the storeys of the models when subjected to single and multiple earthquake. The interstorey drift ratio (IDR) also being evaluated as seismic performance.

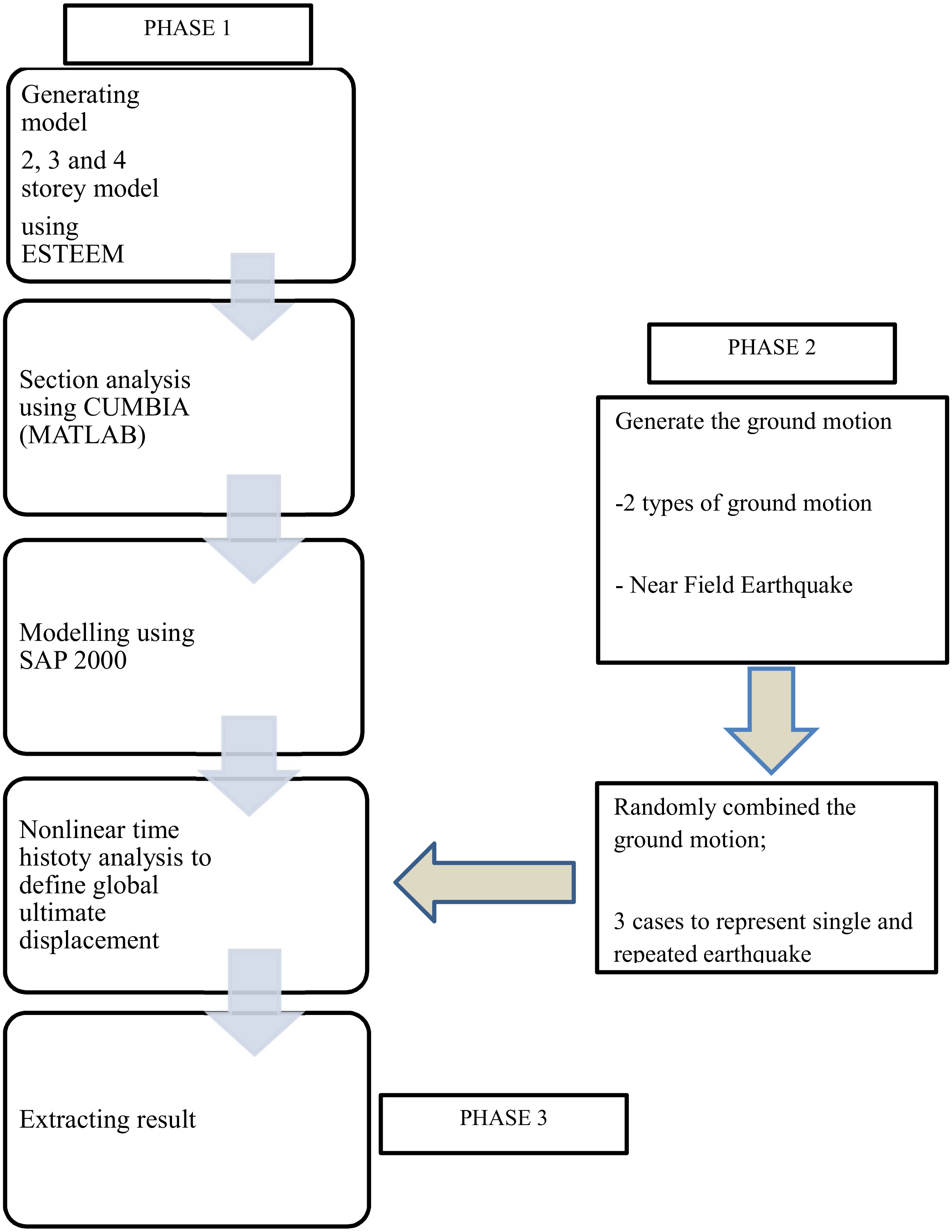
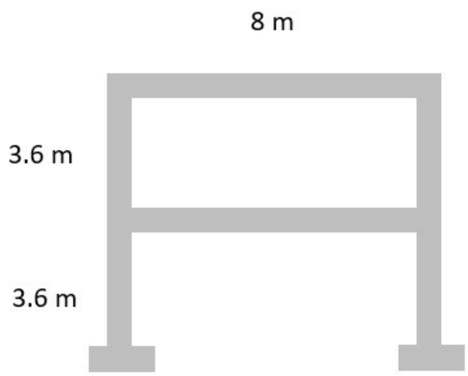


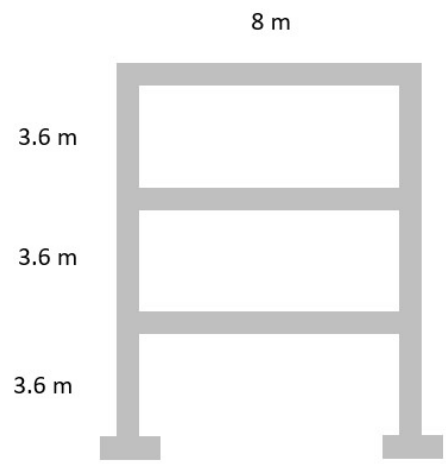
Figure 3.1 Flowchart of research methodology.

3.2 Description of 2D Generic RC School Building

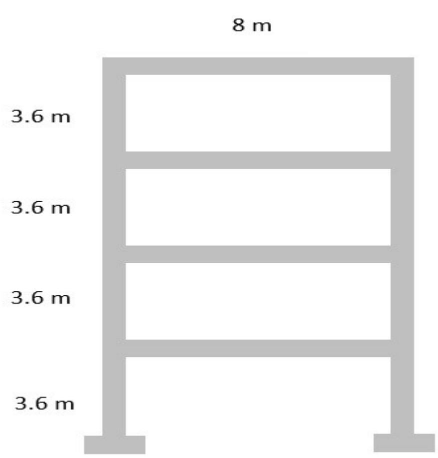
In this study, the 2D multi-storey and single-bay with rigid floor and roof is used. This model is adopted from the typical RC school design in Sabah, Malaysia. In this study, three models of 2 storeys, 3 storeys and 4 storeys represent the low rise buildings. The high rise is neglected since such structures are not suitable for school buildings due to ineffective evacuation process during emergency. The shapes of the model are rectangular in plan B x H with 3.6 meter storey height and a beam that at span B and H are 4.0 meter and 8.0 meter respectively. All models are regular in plan and elevation. Figure 3.2 shows the elevation view of 2D generic models.



(a) Two storey frame model



(b) Three storey frame model



(c) Four storey frame model

Figure 3.2 Elevation view of 2D generic models

The model had negligible torsion effect and to represent this, the mass is placed at the geometric centre of the structure at floor and roof level. In this study, only single bay and multi-storey frames are used as models to save computational effort and reducing the time for analyses. Such approach is practical because their main concern is to study the structural performance only.

In this study, all models have been designed for school classroom which is classified as Category A in Eurocede 1 (2002). Therefore, the live load, Q_k imposed on floor and roof (accessible for maintenance and repairing job only) is equal to 2.0 kN/m^2 and 0.4 kN/m^2, respectively. Weight of materials which contributes to dead load, G_k is shown in Table 3.1 as proposed by Mc Kenzie (2004) and Arya (2009). As mentioned in Chapter 1, this study considers only one type of soil namely as Soil Type D which represent the soft soil referring to Eurocode 8 (2004).

Table 3.1 Weight of materials (Mc Kenzie, 2004)

Material	Weight	Unit
Concrete	24.0	kN/m^3
Finishing	1.0	kN/m^2
Water proofing	0.5	kN/m^2
Suspended ceiling	0.15	kN/m^2
Mechanical and electrical	0.30	kN/m^2
Brickwall	3.0	kN/m^2/m height

The RC design has been performed based on BS8110 (1997) which is currently used for buildings design in Malaysia. The concrete compressive strength, f_{cu}=30 N/mm^2 and yield strength of steel, f_y= 460 N/mm^2. For the two storey buildings, the size of all column is equal to 400 mm x 400 mm while the size of beam at the first and second storey is equal to 250mm x 500 mm. The three storeys model also designed with same code with the size of all column and all beam is equal to 400mm x400mm and 250mm x 550mm respectively. Lastly for the four storey model, the size of column is varying according to the storey level. At level one, the column size is equal to 450mm x 450mm. Level two and three has the size of column equal to 400mm x400m and lastly at the top level which is level four, the size of column is equal to 350mm x 350mm. Nevertheless, the size of beam for all of the storey is similar which is 250mm x 550mm. Therefore, a total of 3 frames has been produced as model for nonlinear time history analysis.

3.3 Fundamental Period of Vibration, T_1

According to Eurocode 8 (2004), the fundamental period of vibration, $T1$ of a structure can be estimated by using the following equation:

$$T_1 = C_t \cdot H^{3/4} \tag{3.1}$$

where H is the total height of the building, in meter, from the foundation or from the top of a rigid basement. For RC moment resisting frame, the coefficient value of C_t in Equation (3.1) shall be taken as 0.075 as stated by Clause 4.3.3.2.2 (3) in Eurocode 8 (2004)

3.4 Design Response Spectrum

From Equation (3.1) in previous section, it is clear that in order to determine the base shear force, F_b acting on the building, the ordinate of the design spectrum at period T_1, $S_d(T_1)$ is required. For this purpose, a series of design response spectrum had been developed as proposed by Clause 3.2.2.5 in Eurocode 8 (2004). This study considers the Type 1 response spectrum which compatible for Soil Type D for seismic hazard in East Malaysia. Equations (3.2) to (3.5) below had been referred to develop the design response spectrum.

$$0 \leq T \leq T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3}\right)\right] \tag{3.2}$$

$$T_B \leq T \leq T_C : S_d(T) = a_g \cdot S \cdot \frac{2.5}{q} \tag{3.3}$$

$$T_C \leq T \leq T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C}{T}\right] \\ \geq \beta \cdot a_g \end{cases} \tag{3.4}$$

$$T_D \leq T : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C T_D}{T^2}\right] \\ \geq \beta \cdot a_g \end{cases} \tag{3.5}$$

where:

T = vibration period of a linear single-degree-of-freedom system

a_g = design ground acceleration on Type A ground ($a_g = \gamma_I . a_{gR}$)

T_B = lower limit of the period of the constant spectral acceleration branch

T_C = upper limit of the period of the constant spectral acceleration branch

T_D = beginning of the constant displacement response range of the spectrum

S = soil factor

q = behaviour factor

$S_d(T)$ = design spectrum

β = lower bound factor for the horizontal design spectrum (0.2)

According to Eurocode 8 (2004), the value of soil factor, S lower limit of the period of the constant spectral acceleration branch, T_B upper limit of the period of the constant spectral acceleration branch, T_C and beginning of the constant displacement response range of the spectrum, T_D are given based on soil type as shown in Table 3.2.

Table 3.2 Main parameters to develop Type 1 design response spectrum (Eurocode 8, 2004)

Soil Type	S	T_B(s)	T_C(s)	T_D(s)
A	1	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	1.5	0.1	0.25	1.2
D	1.8	0.1	0.3	1.2
E	1.6	0.05	0.25	1.2

3.5 Design Ground Acceleration, a_g

By referring to Clause 3.2.1 (3) in Eurocode 8 (2004), the value of design ground acceleration on ground Type A, ag can be determined as follow:

$$a_g = \gamma_I . a_{gR} \tag{3.6}$$

where γ_I and a_{gR} correspond to importance factor and reference peak ground acceleration, respectively. The value of importance factor, γ_I is depends on the importance classes of buildings. In Clause 4.2.5, the Eurocode 8 (2004) classify buildings into four importance classes which depend on the consequences of collapse for human life, importance for public safety, and civil protection as shown in Table 3.3.

The classification also had been made based on consequences of collapse on the social and economic aspects. For example, hospital building had been classified as importance class IV where the integrity of the building during earthquakes is of vital importance for civil protection. Therefore, the value of importance factor, γ_I is equal to 1.4. According to Fardis et al. (2015) the recommended importance factor, $\gamma_I$is to offer better protection of life for such buildings due to its importance after disaster. In this study, the value of importance factor, γ_I is equal to 1.2 because the school buildings has been categorized in important class III.

Table 3.3 Importance classes and importance factors for buildings(Eurocode 8, 2004)

Importance class	**Buildings**	**Importance factor**
I	Buildings of minor importance for public safety, e.g. agricultural buildings. etc.	0.8
II	Ordinary buildings, not belonging in the other categories	1
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse. E.g. school assembly halls, cultural institution etc.	1.2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection. E.g. hospitals, fire stations, power plants, etc.	1.4

Since this study focus on Malaysian seismic hazard, the value of reference peak ground acceleration, a_{gR} assigned in Equation (3.6) is based on Peak Ground Acceleration (PGA) for Malaysia as proposed by Adnan et al. (2008) and also published by MOSTI (2009). The seismic hazard map referred in this study is shown in Figure 3.3 and Figure 3.4 for Peninsular and Eastern Malaysia, respectively. In both Figures, the value of PGA is in unit gal, where 1 gal is equal to 0.001g. Therefore, from Figure 3.3 and Figure 3.4 the value of PGA for Peninsular and Eastern Malaysia is lies in range of 20 gals to 120 gals which is equal to 0.02g to 0.12g. Thus, to represent the moderate seismic region in Sabah, Malaysia the reference peak ground acceleration, a_{gR} equal to 0.12g had been used to develop the design response spectrum.

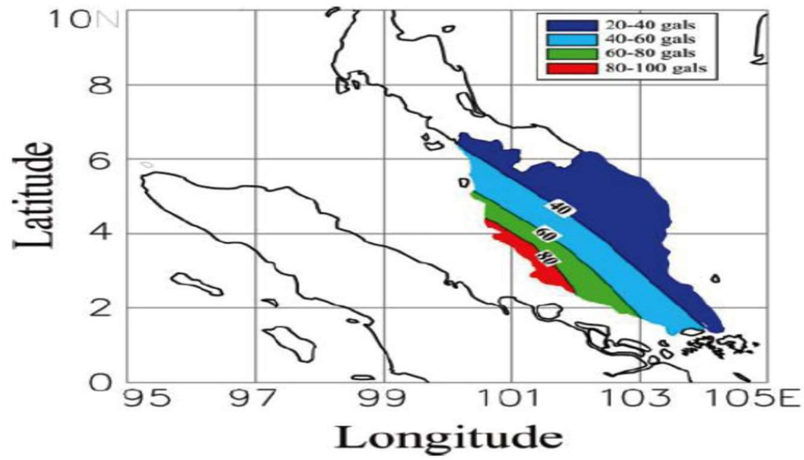


Figure 3.3 Seismic hazard map for Peninsular Malaysia (MOSTI, 2009)

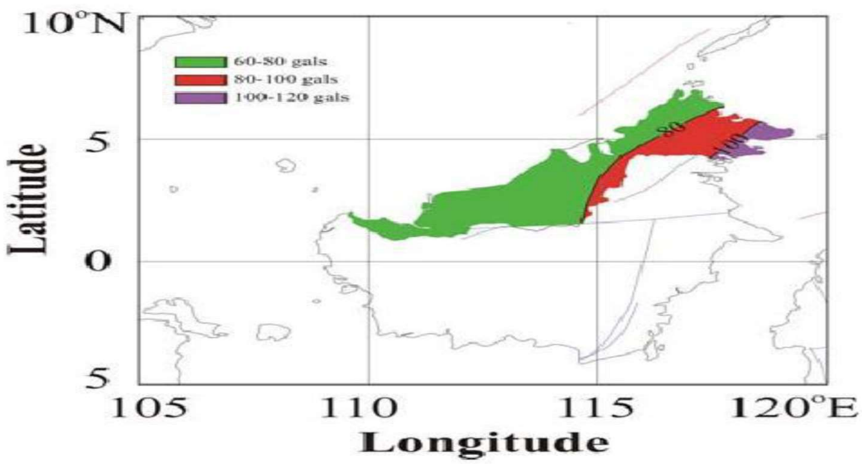


Figure 3.4 Seismic hazard map for Eastern Malaysia (Adnan et al., 2009)

3.6 Ground Motion Records

In this study, a total of 7 real NFE was selected from all over the world. All the ground motion records are taken from PEER database which is selected from earthquake with magnitude equal or more than M_w.5.5. It is done because the design spectrum has been developed based on the design spectrum for Type 1 in Eurocode 8 (2004). Since this study only focus on the effect of single and multiple earthquakes on buildings built on Soil Type D, all the selected ground motion records has been recorded on Soil Type D. Table 3.4 shows the list of selected NFE ground motion records.

Table 3.4 List of selected NFE Ground Motion Records.

No	Event	Component	Station	PGA (g)	PGV (cm/s)	PGD (cm)	Mw
1	Imperial Valley	H-E03140	5057 El. Centro Array #3	0.266	46.8	18.95	6.5
2	Imperial Valley	H-E03230	5057 El. Centro Array #3	0.221	39.9	29.29	6.5
3	Loma Prieta	A02043	1002 APEEL2 Redwood City	0.274	53.6	12.53	6.9
4	Loma Prieta	A02123	1002 APEEL2 Redwood City	0.22	34.3	6.83	6.9
5	Chi-Chi	CHY026=N	CHY026	0.066	32.6	26.99	7.6
6	Chi-Chi	CHY026-W	CHY026	0.076	46.2	35.25	7.6
7	Chi-Chi	CHY032-W	CHY032	0.088	26.4	17.77	7.6

3.6.1 Seismic Sequence

The selected NFE ground motion records were scaled by using Matlab program to match the design response spectrum of Sabah as proposed by Eurocode 8(2004). Table 3.5 shows the scale factor of all NFE ground motion records. Next, to generate the artificial multiple earthquakes, the ground motion records were combined randomly by using Matlab program. For this purpose, the random combination of the forceshock and aftershock has been randomly generated using random functions in Ms Excel according to the number of ground motion recorded in Table 3.4

Table 3.5 Scaling of real near field earthquake to response spectrum

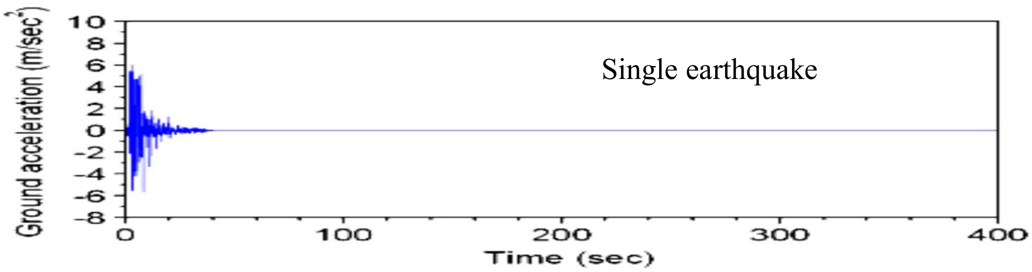
Fundamental period of vibration, T_1 = 0.24 sec (N=2)			
No.	**Sa(T_1), g**		**Scale Factor**
	Original Record	**Response Spectrum**	
DNFS1	0.8472	0.486	0.574
DNFS2	0.6440	0.486	0.755
DNFS3	0.3273	0.486	1.485
DNFS4	0.2475	0.486	1.9780
DNFS5	0.1214	0.486	4.0033
DNFS6	0.1594	0.486	3.0489
DNFS7	0.1757	0.486	2.7660

All ground motion records use in the analysis are divided into two types of motion. For motion 1, only one ground motion which is only the mainshock. This is called single earthquake. Whereas the motion 2 include the three ground motion (foreshock-mainshock – aftershock). Table 3.6 shows the combination of randomly selected single earthquake to form the multiple earthquake.

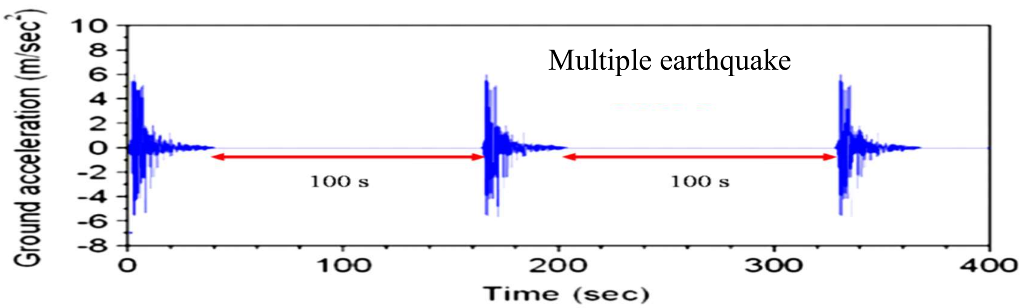
Table 3.6: Combination of single earthquake to generate artificial multiple earthquake

NO.	**FORESHOCK**	**MAINSHOCK**	**AFTERSHOCK**
DNFR1	DNFS1	DNFS7	DNFS4
DNFR2	DNFS2	DNFS6	DNFS3
DNFR3	DNFS3	DNFS5	DNFS2
DNFR4	DNFS4	DNFS4	DNFS1
DNFR5	DNFS5	DNFS3	DNFS7
DNFR6	DNFS6	DNFS2	DNFS5
DNFR7	DNFS7	DNFS1	DNFS6

Between two consecutive seismic events a time gap is also applied in this case to cease the vibration of any structure due to damping. The time gap used in this method is 100 seconds and this gap has zero acceleration ordinates and is absolutely enough to cease the moving of any structure due to damping as suggested by Hatziegeorgiou and Liolios (2010). Figure 3.5 shows the typical profile of ground motion records for both cases.



(a) Motion 1



(b) Motion 2

Figure 3.5 Typical profile of generated ground motion with 100s gaps.

3.7 Section Analysis by Using Cumbia

In order to run the nonlinear time history analysis, the nonlinear properties of all structural elements, which is beams and columns of all frames has to be determined through simple process namely as section analysis. In this study, the section analyses were performed by using CUMBIA program (Montejo and Kowalsky, 2007). CUMBIA program is a set of MATLAB code used to perform monotonic moment curvature analysis of RC members. The software needs Matlab (2014) software to run the analysis. Before run the section analysis, the input in term of structural geometry and its steel reinforcement had to be assigned alongside the strength of materials. The outcome for this method is to define moment-curvature and moment-axial interaction by using CUMBIA program. For section analysis, it is run for both beams and columns having different sizes of reinforcement bars and cross sections.

Firstly, the software MATLAB R2006a is opened. “File” is selected, then “open” is clicked. Either beam or column is chosen for analysis. After the file is open, the file name is inserted. “n” is inserted for interaction. For the section properties, the section height (mm), section width (mm), quantity of transverse steel in x-direction (confinement), quantity of transverse steel in y-direction (shear), cover to longitudinal bars (mm) is specified. In member properties, member clear length (mm,) bending (single or double) and ductility mode as “uniaxial” is inserted. Distance of rebar is inserted in MLR which is a matrix composed of [distance from the top to bar center (mm) of bars – number of bars - bar diameter (mm)]. Each row corresponds to a layer of reinforcement. For transverse reinforcement details, diameter of transverse reinforcement spacing of transverse steel is specified. Applied load from SAP2000 for column and “0” for beam is inserted. Material properties are inserted as shown below:

fpc = 30; Concrete compressive strength (MPa)

Ec = 26000; Concrete modulus of elasticity (MPa)

eco = 0.002; Unconfined strain

esm = 0.12; Maximum transverse steel strain (usually about 0.10-0.15)

espall = 0.0064; Maximum unconfined concrete strain (usually 0.0064) 32

fy = 460; Longitudinal steel yielding stress (MPa)

fyh = 250; Transverse steel yielding stress (MPa)

Es = 205000; Steel modulus of elasticity

fsu = 600; Longitudinal steel maximum stress (MPa)

esh = 0.008; Longitudinal steel strain for strain hardening

esu = 0.12; Longitudinal steel maximum strain (usually about 0.10-0.15)

Ey = 700; Slope of the yield plateau (MPa)

C1 = 3.3; Defines strain hardening curve in the Raynor model (about 2-6)

After that, “debug” is selected to run the analysis. Several figures will be shown after the process is completed. Figure 3.6 and Table 3.7 show the sample of moment-curvature curve for beam. CUMBIA folder is opened and MS excel file is clicked for either beam or column output. User SF, hinge length, ultimate moment, ultimate curvature, yield moment and yield curvature from CUMBIA are the data specified in SAP2000 for nonlinear analysis.

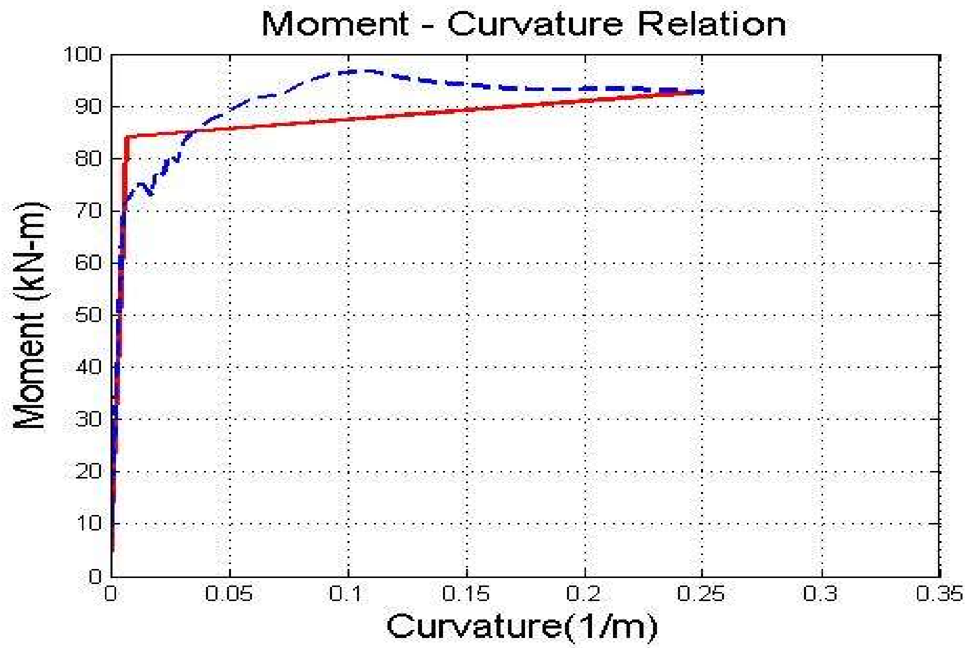


Figure 3.6 Moment-curvature curve (CUMBIA).

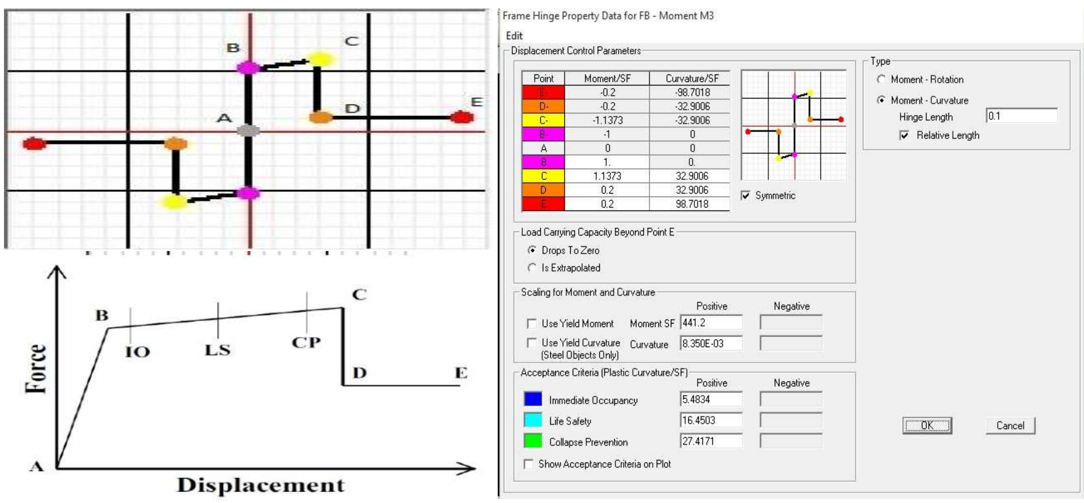
Table 3.7 Moment-curvature data (Cumbia)

Curvature (1/m)	Moment (kNm)	Displacement (m)	Force (kN)
0	0	0	0
0.00686	84.16	0.13628	11
0.25065	92.7	1.22183	12.12

This step is continued for all structural elements which is beams and columns of every models.

3.8 Plastic Hinge Properties at Member's End

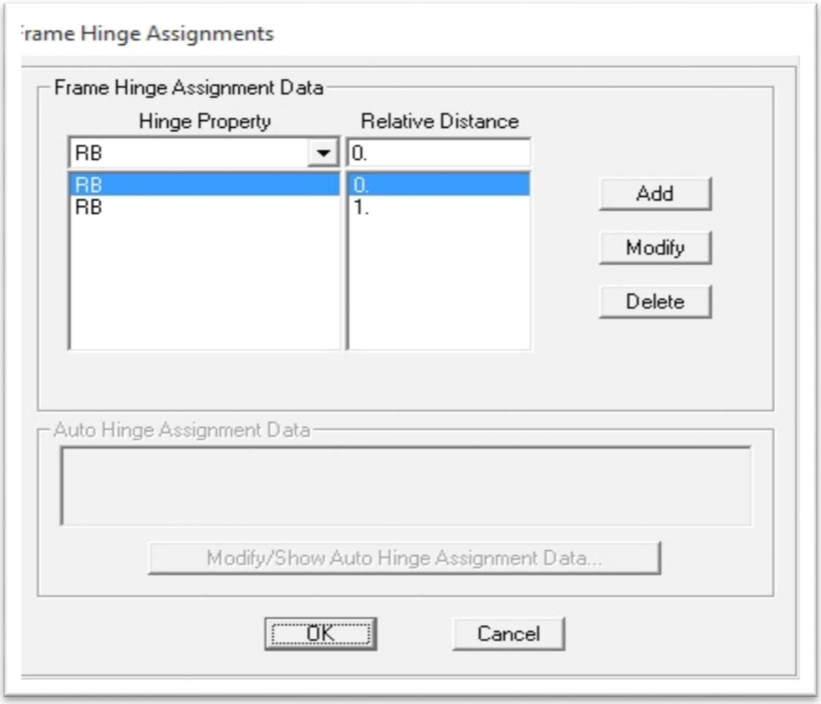
The procedure is continuing with analyzing hinges at members' ends. It depends on the section analysis result data obtained from CUMBIA analysis. Moment-curvature data are used and hence hinge length is assumed to be *0.5H* of member section because it is dealing with RC frame (Park and Paulay, 1975). There is no need of hinge length if moment rotation is selected. This will give a force-displacement curve with strength and deformation points as shown in Figure 3.20. Value used for SAP2000 is the point B-C-D-E. The procedure is repeated for different strength and deformation of structural members.



(a) **(b)**

Figure 3.7 Assigning the hinge property in SAP2000; (a) Force-Displacement Relationship and (b) Defining Frame Hinge Property

The main purpose of plastic hinge defined from CUMBIA section analysis is used to assign the location of plastic hinge properties of the structural elements to the member ends. The plastic hinge is assigned at each member ends which couple with 0 and 1 of relative distance mean the closer and the far ends position as shown in Figure 3.8. Thus, this step is repeated for all structural members of the models.



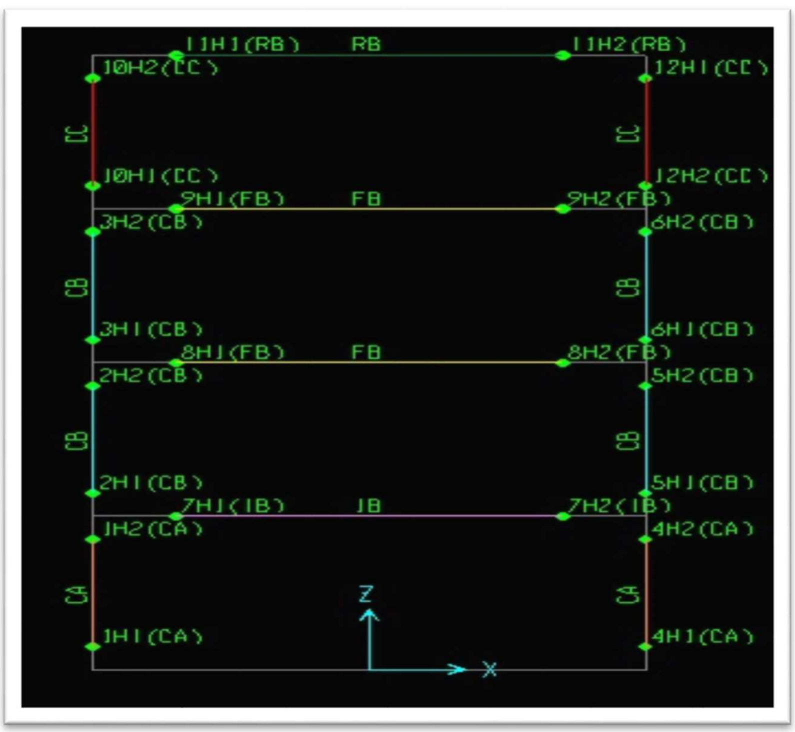


Figure 3.8 Assignment of the plastic hinge to the structural member ends of the elements for N=4 model

3.9 Data and Analysis

In this study, the nonlinear time history analysis has been conducted by employing SAP 2000. By using nonlinear time history analysis in SAP2000, the lateral displacement of every storey for each models is obtained when subjected to single earthquake and multiple earthquake. Thus, the value of IDR of every storey can be obtained for investigating and determine the performance of the buildings when subjected to single earthquake and multiple earthquake

3.10 Summary of Procedure

In general, Table 3.8 shows the parameter that was used in this study. Stet-by-step procedure can be briefly explained as follows.

1. Determine type of building models and design the 2, 3 and 4 storey RC school building models by using ESTEEM sotware to represent the existing buildings.
2. Perform section analysis on the structural members of the models by using CUMBIA program to determine the nonlinear properties and hinge properties of the structural members.
3. Modelling the 2, 3 and 4 storey RC school models by using SAP2000.
4. Selecting the seismic region including the Soil Type in order to develop the design response spectrum.
5. Selecting seven NFE ground motion records from all over the world.
6. Calculating the fundamental period (T_1) based on the total height of the model.
7. Scaling of selected ground motion records to match the design response spectrum by using Matlab and Seismosignals software.
8. Generate seven artificial multiple earthquakes from randomly selected single near field ground motion record by using Matlab.
9. Running nonlinear time history analysis of the RC school models subjected to single and multiple earthquakes by using SAP2000
10. Obtain maximum lateral displacement of the model when subjected to single and multiple earthquake.
11. Calculating the IDR for every storey of the buildings when subjected to single and multiple earthquakes.

12. Repeating step 9 to 11 on all models.

Table 3.8 Parameters used in this study

Parameter	Description
Model	Two, three, and four storey 2D generic frame
Ground Motion	Near Field Earthquake (NFE)
No of ground motion	7 real ground motions records
Ground motion combination	Motion 1: single earthquake (mainshock) Motion 2: repeated earthquake (foreshock-mainshock-aftershock)
Respond parameter considered	Maximum lateral displacement (m) Interstorey drift ratio (%)
Analysis	Linear Static analysis (Modal Analysis) Nonlinear Time History Analysis
Main software	SAP 2000 ESTEEM8 Matlab (CUMBIA) Seismosignals

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Introduction

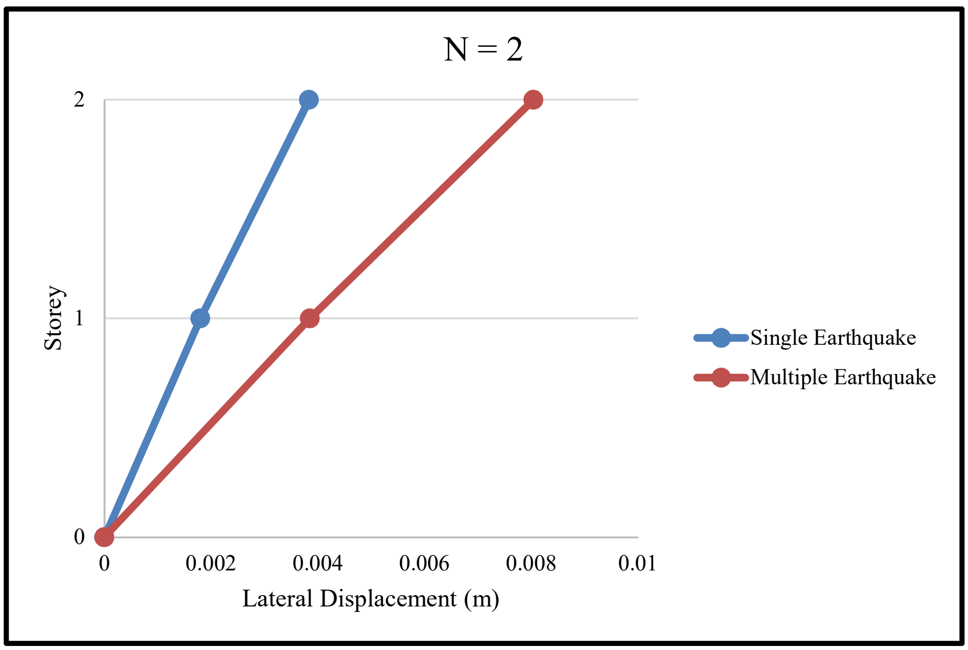
In this chapter, the result and discussion based on analyses performed is presented. In order to study the nonlinear behaviour of structural system, it is important to perform the nonlinear time history analysis which clearly simulates the models with ground motion records. The non-linear time history analysis on three RC frames (2, 3 and 4 storey models) has been conducted by using SAP2000 program. The result of nonlinear time history analysis in term of interstorey drift ratio (IDR) of both single and multiple earthquakes are discussed in this chapter.

4.2 Nonlinear Time History Analysis

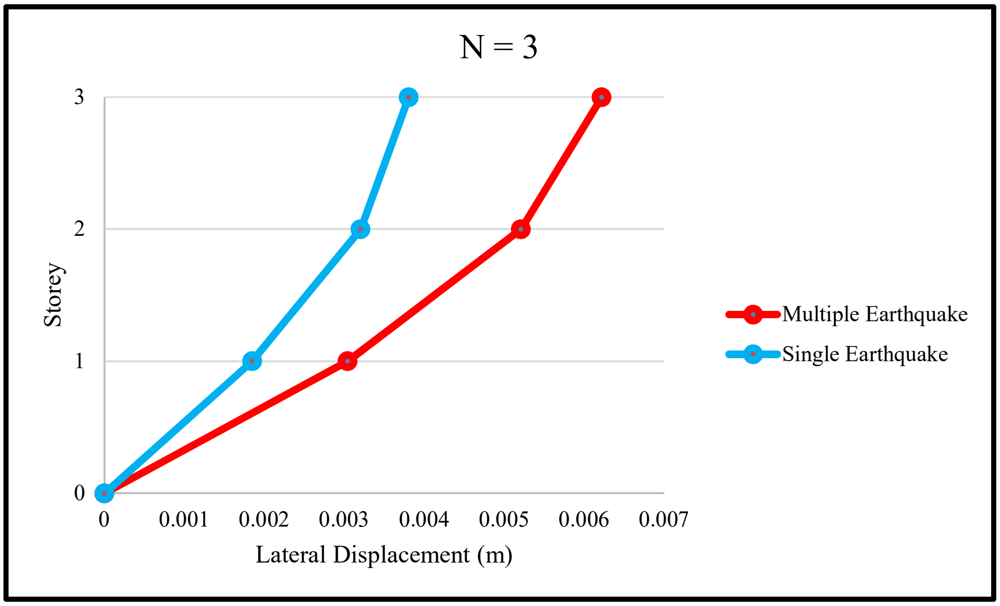
4.2.1 Lateral Displacement

From the nonlinear time history analysis, the result obtained was maximum lateral displacement of each storey and roof for 2, 3 and 4 storey models when subjected to 7 single and 7 multiple earthquakes respectively. The result of the maximum lateral displacement (in meter) for every storey for each model is shown on Appendix A for both single earthquake and multiple earthquakes.

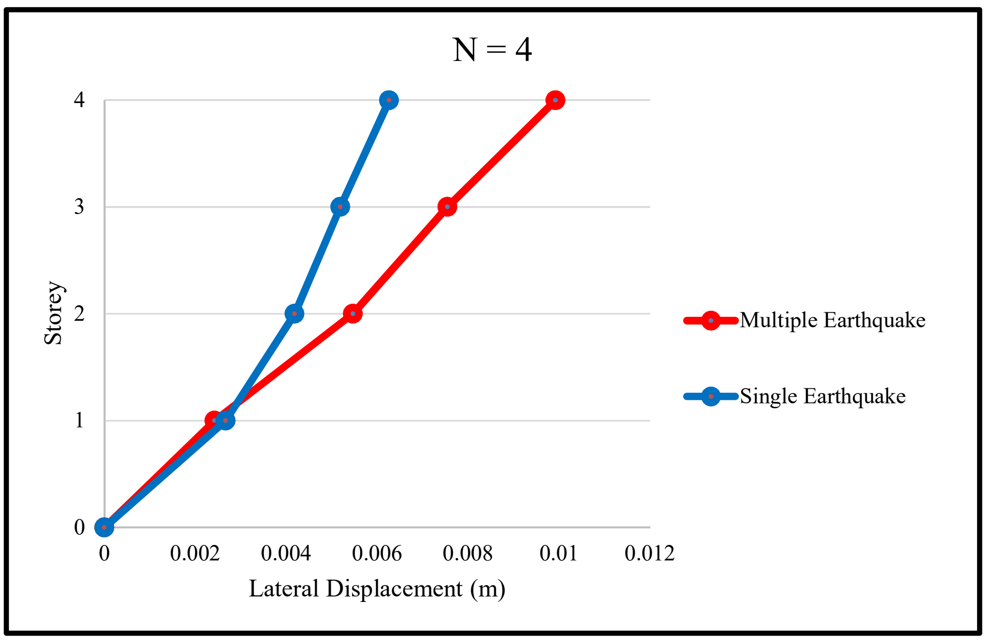
By referring to appendix A, the value of lateral displacement of every storeys for every models when subjected to single and multiple earthquake which has been analysed by using nonlinear time history analysis in SAP2000 was recorded. For the two storey model (N=2), the highest value of lateral displacement is caused by DNFS3 which contribute 0.00771 m displacement at top floor while the smallest lateral displacement was caused by DNFS7 which is 0.0005m. Thus, to ensure the precise and accuracy of this study, the author use mean value of lateral displacement which has to be taken from all 7 ground motion records. This method has been proposed by Eurocode 8 (2004), in order to increase the precision and accuracy of the result. Therefore, mean lateral displacement from every model when subjected to single and multiple earthquake has been considered. The distribution of lateral displacement and mean lateral displacement when subjected to single earthquake and multiple earthquake is presented in Appendix B. Thus for comparison and discussion of the result purposes, the mean lateral displacement for both single and multiple earthquakes is plotted on the same axis graph. Figure 4.1 presents the graph of mean lateral displacement (in meter) of every models when subjected to single and multiple earthquakes.



(a) Mean lateral displacement of single and multiple earthquake for N=2



(b) Mean lateral displacement of single and multiple earthquake for N=3



(c) Mean lateral displacement of single and multiple earthquake for N=4

Figure 4.1 Mean lateral displacement (m).

Figure 4.1 (a) presents the mean lateral displacement of double storey model (N=2). It can be seen that the maximum lateral displacement caused by multiple earthquake is equal to 0.008m at the top level. While, the maximum mean lateral displacement caused by single earthquake is equal to 0.003m also at the top level. Thus, the mean lateral displacement caused by multiple earthquake is relatively 72.73% higher compared to single earthquake.

By referring to Figure 4.1 (b), the similar pattern has shown on lateral displacement caused by single and multiple earthquake for 3 storey model (N=3). For this model, the maximum mean lateral displacement is equal to 0.0062m where single earthquake only shows 0.0038m. This shows that the multiple earthquake contribute 62% higher lateral displacement compared to the single earthquake subjected to the model. The 62% higher lateral displacement of the single earthquake occasionally contribute greater damage to the structural element of the structure. Thus, it is important to consider multiple earthquake in design of anti-seismic load structure.

Figure 4.1 (c) presents the mean lateral displacement (in meter) for the 4 storey model (N=4) when subjected to single and multiple earthquakes. It can be seen that the maximum value of mean lateral displacement caused by multiple earthquake event is equal to 0.009m at the top floor, while, single earthquake shows the maximum value of mean lateral displacement equal to 0.00625m. Thus, lateral displacement caused by multiple earthquake is relatively around 58.06% higher compared to single earthquake. This result was in line with the previous study discussed by Hatzigeorgiou (2010b) which concluded that the multiple earthquake requires high demand on lateral displacement compared to single earthquake. This is affected by the stiffness and strength degradation occurred to the structural elements after being hit by the first tremor. Hence, when not yet repaired structure subjected to second and third tremors, it generally exposed to higher lateral displacement.

4.2.2 Interstorey Drift Ratio

The results of the maximum percentage of IDR for every models for single and multiple earthquakes is attached in Appendix C. In this part, 7 numbers of single earthquake named as DNFS1 until DNFS7 as well as 7 artificial ground motion generated from random combination of single ground motion to represent the multiple earthquake namely DNFR1 until DNFR7 has been used in nonlinear time history analysis.

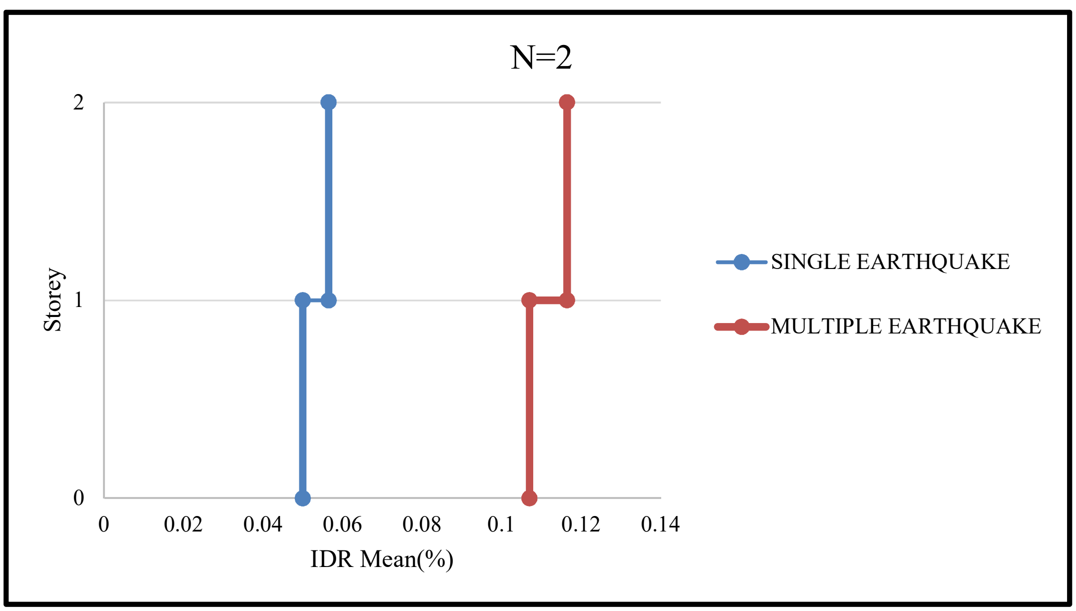
In this study, the seismic performance of each RC school model is evaluated based on the IDR (%). Generally, it is used to determine and evaluate the performance of the RC school buildings when subjected to single earthquake and multiple earthquake. IDR is the relative horizontal displacement between two adjacent storey normalized to its storey height. Basically, in Performance-Based Earthquake Engineering, there are four performance level namely as Operational (OP), Immediate Occupancy (IO), Life Safety (LS), and Near Collapse (NC) that might be experienced by structures during earthquake. In addition, the mean value of IDR, denotes as IDR_{mean} (%) is determined from 7 numbers of single and multiple earthquake. This method is used as proposed by Eurocode 8 (2004) in order to increase the accuracy and precision of the results. Thus the IDR_{mean} is considered in this study for discussion and comparison purpose. The distribution of IDR and IDR_{mean} for every models when subjected to single earthquake and multiple earthquakes is presented in Appendix C.

Figure 4.2 (a) presents the IDR_{mean} of double storey model (N=2). It can be seen that the maximum IDR_{mean} caused by multiple earthquake is equal to 0.116 % at the top level. While, the maximum IDR_{mean} caused by single earthquake is equal to 0.056% also at the top level. Thus, the IDR_{mean} caused by multiple earthquake is relatively 67.44% higher compared to single earthquake.

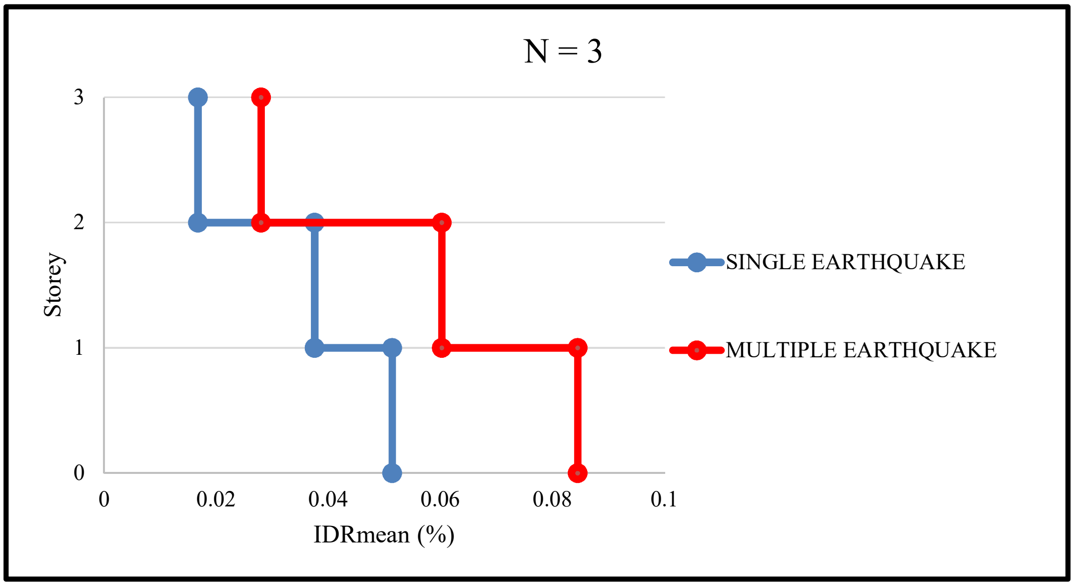
By referring to Figure 4.2 (b), the similar pattern has shown on IDR_{mean} caused by single and multiple earthquake for 3 storey model (N=3). For this model, the maximum IDR_{mean} is equal to 0.084% which occurs at level two, while single earthquake only shows 0.051%. This shows that the multiple earthquake contributes 62.2% higher IDR_{mean} compared to the single earthquake subjected to the model. The 62.2% higher

IDR_{mean} of the single earthquake occasionally contribute greater damage to the structural element of the structure. Thus, it is important to consider multiple earthquake in design of anti-seismic load structure.

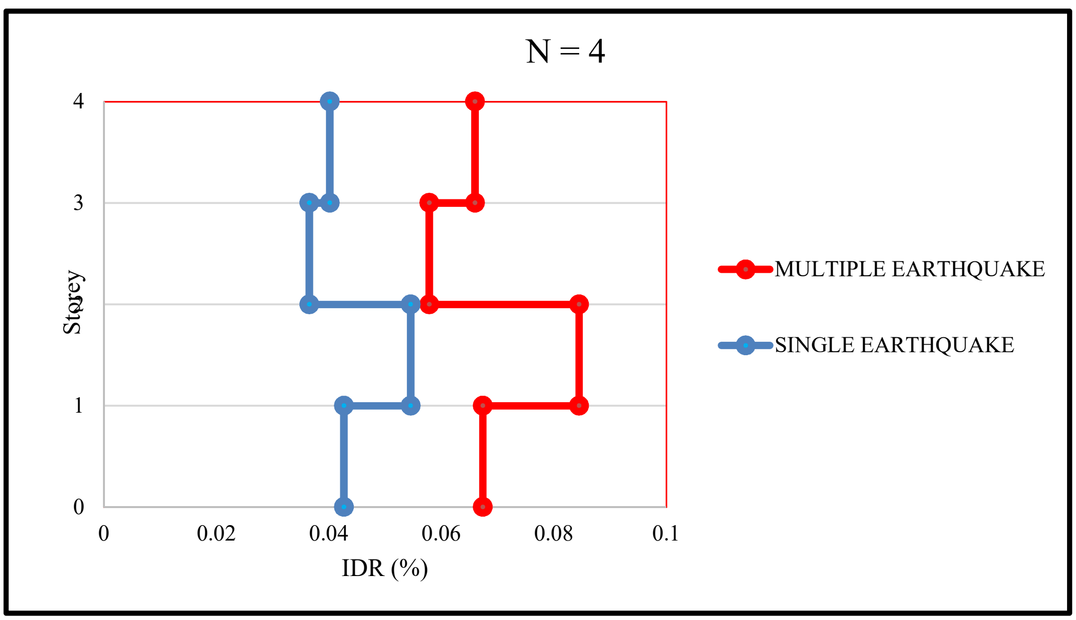
Figure 4.2 (c) presents the IDR_{mean} of the four storey model (N=4) when subjected to single and multiple earthquakes. It can be seen that the value of maximum IDR_{mean} caused by multiple earthquake event is equal to 0.084% while maximum value of IDR_{mean} for single earthquake is equal to 0.0545%. Thus, IDR_{mean} caused by multiple earthquake is relatively 60.65% higher compared to single earthquake. This result was in line with the previous study discussed by Hatzigeorgiou (2010b) which conclude that the multiple earthquake requires high demand on IDR_{mean} compared to single earthquake. This is affected by the stiffness and strength degradation occurred to the structural elements after hit by the first tremor. Hence, when not yet repaired structure subjected to second and third tremors, it generally requires higher demand of IDR. This result also is in line with Hatzigeorgiou and Liolos (2010) and Adiyanto and Majid (2014).



(a) Mean Interstorey Drift Ratio for N=2



(b) Mean Interstorey Drift Ratio for N=3



(c) Mean Interstorey Drift Ratio (IDR_{mean}) for N=4

Figure 4.2 Mean Interstorey Drift Ratio (%)

CHAPTER 5

CONCLUSION

5.1 Conclusion

The objective of this study are to determine the seismic performance of reinforced concrete (RC) school buildings in Sabah, Malaysia when subjected to single and multiple earthquakes. The seismic performance was evaluated based on lateral displacement and interstorey drift ratio (IDR) of each storey of the models when subjected to single and multiple earthquakes. To achieve these objectives, three types of (RC) school model has been used. The model is assumed to be constructed on Soil Type D to represent the moderate seismicity region of Sabah based on Malaysia Earthquake Hazard Map. The model was designed based on BS8110 (1997) to represent existing RC school in Sabah. 7 numbers of near field earthquake ground motion records has been selected from all over the world as single earthquake. Furthermore, 7 numbers of artificial multiple earthquake has been generated by using combination of three single ground motion namely as foreshock, mainshock and aftershock. All those ground motion records were used to run nonlinear time history analysis by using SAP2000 program. The conclusions reached from this study are listed as follows.

- The value of lateral displacement of the building is higher when subjected to multiple earthquake compared to single earthquake. Based on nonlinear time history analysis, the value of maximum lateral displacement of every storey due to multiple earthquakes is around 58% - 73% higher compared to single earthquake. Thus, multiples earthquakes action cause more structural and non-structural damage to the buildings compared to single earthquake.
- The action of multiple earthquake ground motion on the models contribute higher value of IDR compared to the single one. In this study, the action of multiple

earthquake has contributed around 60% - 67% higher IDR compared to the single earthquake. Thus, multiple earthquake provision should be taken in order to design new buildings as well as the evaluation for maintenance and rehabilitation for existing buildings.

5.2 Recommendation

For future enhancement of this study, the following areas of investigation are recommended:

i. This study has examined 2, 3 and 4 storey buildings which is considered as low rise buildings. Similar study on tall buildings can be conducted since the effect of earthquake on high rise buildings is more obvious.
ii. Since this study only consider symmetrical frame, further investigation for building with plan irregularity, stiffness irregularity, mass irregularity or combination of different types of irregularity should be carried out.

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APPENDIX A
MAXIMUM LATERAL DISPLACEMENT

SINGLE EARTHQUAKE

Table A1a: Lateral displacement (m) of N=2 when subjected to single earthquake

LEVEL	LATERAL DISPLACEMENT (m)							
	DNFS1	DNFS2	DNFS3	DNFS4	DNFS5	DNFS6	DNFS7	MEAN
2	0.00771	0.00771	0.00771	0.00069	0.0009	0.0016	0.0005	0.003831
1	0.00364	0.00364	0.00364	0.0003	0.0004	0.0007	0.00028	0.0018
0	0	0	0	0	0	0	0	0

Table A1b: Lateral displacement (m) of N=3 when subjected to single earthquake

LEVEL	LATERAL DISPLACEMENT (m)							
	DNFS1	DNFS2	DNFS3	DNFS4	DNFS5	DNFS6	DNFS7	MEAN
3	0.00609	0.00648	0.00598	0.00599	0.0007	0.0009	0.0005	0.003806
2	0.00501	0.00538	0.00511	0.00512	0.0006	0.0008	0.0004	0.003203
1	0.0028	0.00305	0.00309	0.00311	0.0003	0.0004	0.0002	0.00185
0	0	0	0	0	0	0	0	0

Table A1c: Lateral displacement (m) of N=4 when subjected to single earthquake

STOREY	LATERAL DISPLACEMENT (m)							
	DNFS1	DNFS2	DNFS3	DNFS4	DNFS5	DNFS6	DNFS7	MEAN
4	0.01018	0.00998	0.00976	0.00928	0.00131	0.00277	0.000535	0.00625929
3	0.01018	0.00744	0.0076	0.00775	0.00098	0.00196	0.000393	0.00518614
2	0.01018	0.00536	0.00563	0.0058	0.0007	0.00133	0.00028	0.00418286
1	0.01018	0.0023	0.00256	0.00268	0.000293	0.00052	0.00013	0.00266614
0	0	0	0	0	0	0	0	0

MULTIPLE EARTHQUAKES

Table A2a: Lateral displacement (m) of N=3 when subjected to multiple earthquake

LEVEL	LATERAL DISPLACEMENT (m)							
	DNFR1	DNFR2	DNFR3	DNFR4	DNFR5	DNFR6	DNFR7	MEAN
2	0.00837	0.00716	0.00837	0.0078	0.0085	0.0084	0.0077	0.008043
1	0.00395	0.00356	0.00395	0.0039	0.004	0.004	0.0036	0.003851
0	0	0	0	0	0		0	0

Table A2b: Lateral displacement (m) of N=3 when subjected to multiple earthquake

LEVEL	LATERAL DISPLACEMENT (m)							
	DNFR1	DNFR2	DNFR3	DNFR4	DNFR5	DNFR6	DNFR7	MEAN
3	0.00648	0.00598	0.00648	0.00599	0.006	0.0065	0.0061	0.006219
2	0.00538	0.00511	0.00538	0.0051	0.0051	0.0054	0.005	0.00521
1	0.00305	0.00309	0.00305	0.0031	0.0031	0.0031	0.0028	0.003041
0	0	0	0	0	0	0	0	0

Table A2c: Lateral displacement (m) of N=4 when subjected to multiple earthquake

	LATERAL DISPLACEMENT (m)							
LEVEL	DNFR1	DNFR2	DNFR3	DNFR4	DNFR5	DNFR6	DNFR7	MEAN
4	0.00998	0.00976	0.00998	0.00982	0.00976	0.00998	0.01018	0.00992286
3	0.00744	0.0076	0.00744	0.00775	0.0076	0.00744	0.00757	0.00754857
2	0.00536	0.00563	0.00536	0.0058	0.00536	0.00536	0.00539	0.00546571
1	0.0023	0.00256	0.0023	0.00268	0.00256	0.0023	0.00227	0.00242429
0	0	0	0	0	0	0	0	0

APPENDIX B
DISTRIBUTION OF MAXIMUM LATERAL DISPLACEMENT

SINGLE EARTHQUAKE

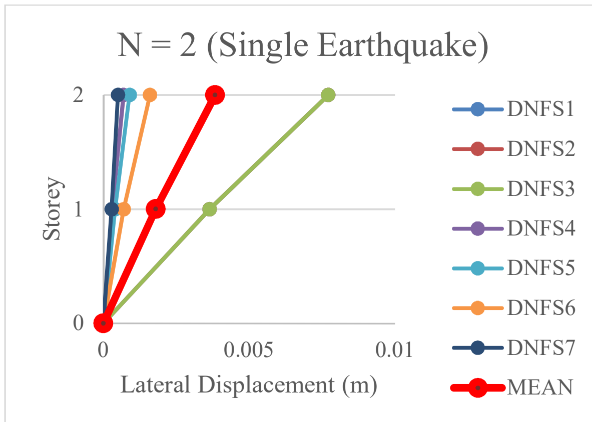

Figure B1a: Lateral displacement (m) of single earthquake for N=2

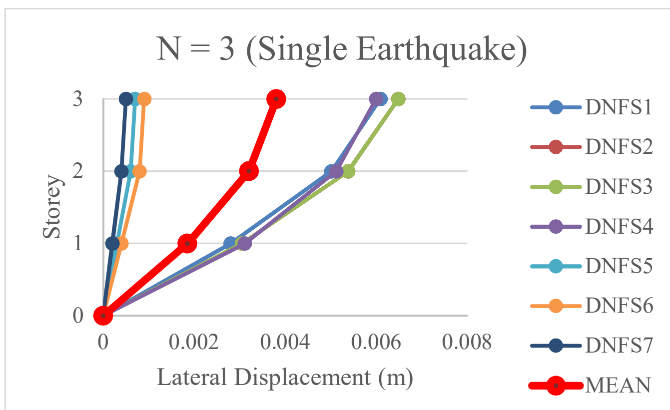

Figure B1b: Lateral displacement (m) of single earthquake for N=3

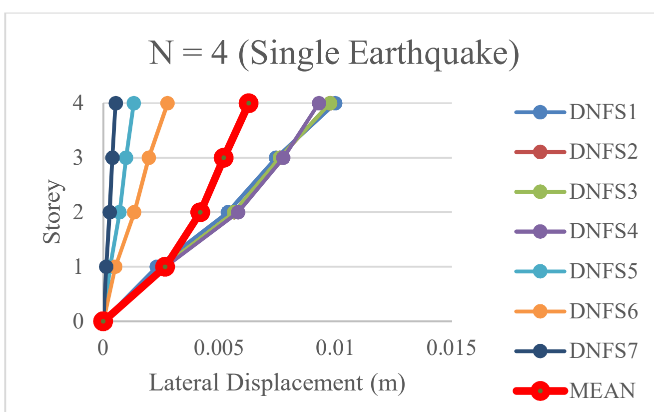

Figure B1c: Lateral displacement (m) of single earthquake for N=4

MULTIPLE EARTHQUAKES

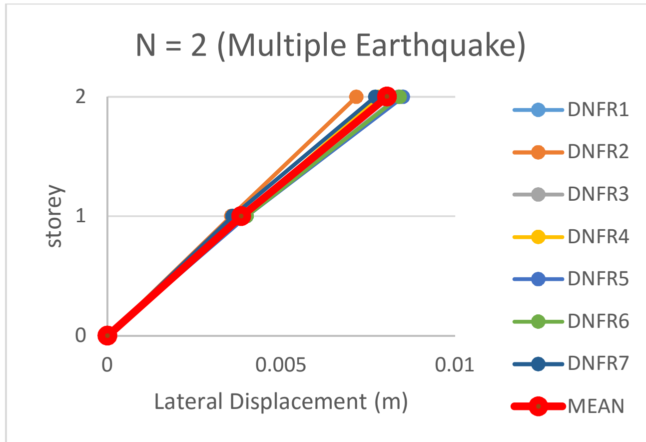

Figure B2a: lateral displacement (m) of multiple earthquake for N=2

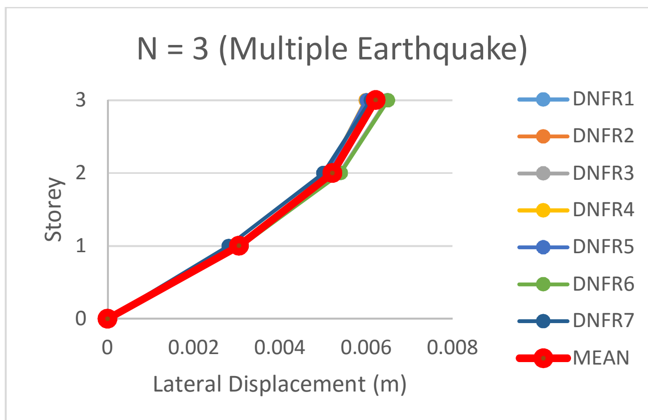

Figure B2b: lateral displacement (m) of multiple earthquake for N=3

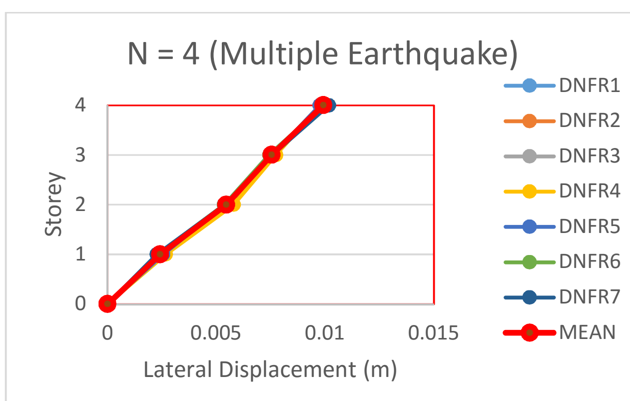

Figure B2c: lateral displacement (m) of multiple earthquake for N=4

APPENDIX C
INTERSTOREY DRIFT RATIO

SINGLE EARTHQUAKE

Table C1a: Interstorey Drift Ratio (%) of N=2 when subjected to single earthquake

LEVEL	INTERSTOREY DRIFT RATIO (%)							
	DNFS1	DNFS2	DNFS3	DNFS4	DNFS5	DNFS6	DNFS7	MEAN IDR
2	0.113056	0.113056	0.113056	0.010833	0.013889	0.025	0.006111	0.05642857
1	0.101111	0.101111	0.101111	0.008333	0.011111	0.019444	0.007778	0.05
0	0	0	0	0	0	0	0	0

Table C1b: Interstorey Drift Ratio (%) of N=3 when subjected to single earthquake

LEVEL	INTERSOREY DRIFT RATIO (%)							
	DNFS1	DNFS2	DNFS3	DNFS4	DNFS5	DNFS6	DNFS7	MEAN
3	0.03	0.030556	0.024167	0.024167	0.002778	0.002778	0.002778	0.016746
2	0.061389	0.064722	0.056111	0.055833	0.008333	0.011111	0.005556	0.037579
1	0.077778	0.084722	0.085833	0.086389	0.008333	0.011111	0.005556	0.051389
0	0	0	0	0	0	0	0	0

Table C1c: Interstorey Drift Ratio (%) of N=4 when subjected to single earthquake

LEVEL	INTERSTOREY DRIFT RATIO (%)							
	DNFS1	DNFS2	DNFS3	DNFS4	DNFS5	DNFS6	DNFS7	mean
4	0	0.070556	0.06	0.0425	0.009167	0.0225	0.003944	0.02981
3	0	0.057778	0.054722	0.054167	0.007778	0.0175	0.003139	0.027869
2	0	0.085	0.085278	0.086667	0.011306	0.0225	0.004167	0.042131
1	0.282778	0.063889	0.071111	0.074444	0.008139	0.014444	0.003611	0.07406
0	0	0	0	0	0	0	0	0

MULTIPLE EARTHQUAKES

Table C2a: Interstorey Drift Ratio (%) of N=2 when subjected to multiple earthquake

LEVEL	INTERSTOREY DRIFT RATIO (%)							
	DNFR1	DNFR2	DNFR3	DNFR4	DNFR5	DNFR6	DNFR7	MEAN IDR
2	0.122778	0.1	0.122778	0.108333	0.125	0.122222	0.113889	0.11642857
1	0.109722	0.098889	0.109722	0.108333	0.111111	0.111111	0.1	0.10698413
0	0	0	0	0	0	0	0	0

Table C2b: Interstorey Drift Ratio (%) of N=3 when subjected to multiple earthquake

LEVEL	INTERSOREY DRIFT RATIO (%)							
	DNFR1	DNFR2	DNFR3	DNFR4	DNFR5	DNFR6	DNFR7	MEAN
3	0.030556	0.024167	0.030556	0.024722	0.025	0.030556	0.030556	0.028016
2	0.064722	0.056111	0.064722	0.055556	0.055556	0.063889	0.061111	0.060238
1	0.084722	0.085833	0.084722	0.086111	0.086111	0.086111	0.077778	0.084484
0	0	0	0	0	0	0	0	0

Table C2c: Interstorey Drift Ratio (%) of N=4 when subjected to multiple earthquake

	INTERSTOREY DRIFT RATIO (%)							
LEVEL	DNFR1	DNFR2	DNFR3	DNFR4	DNFR5	DNFR6	DNFR7	mean
4	0.070556	0.06	0.070556	0.0575	0.06	0.070556	0.0725	0.065952
3	0.057778	0.054722	0.057778	0.054167	0.062222	0.057778	0.060556	0.057857
2	0.085	0.085278	0.085	0.086667	0.077778	0.085	0.086667	0.084484
1	0.063889	0.071111	0.063889	0.074444	0.071111	0.063889	0.063056	0.067341
0	0	0	0	0	0	0	0	0

APPENDIX D

DISTRIBUTION OF INTERSTOREY DRIFT RATIO

SINGLE EARTHQUAKE

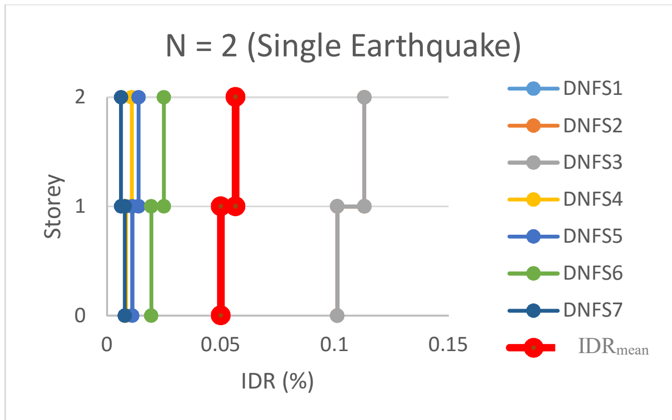

Figure D1a: Interstorey Drift Ratio (%) of single earthquake for N=2

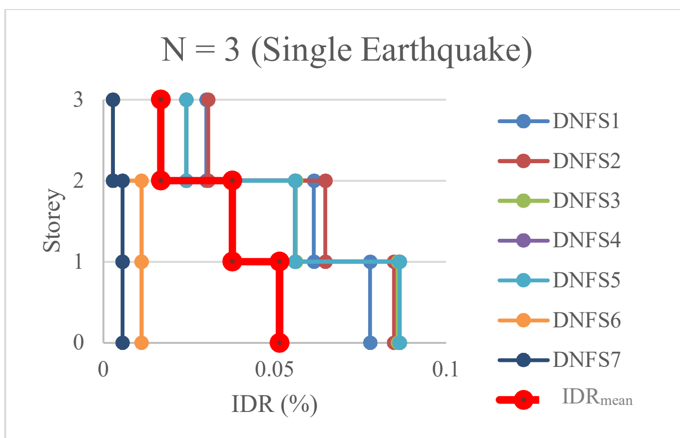

Figure D1b: Interstorey Drift Ratio (%) of single earthquake for N=3

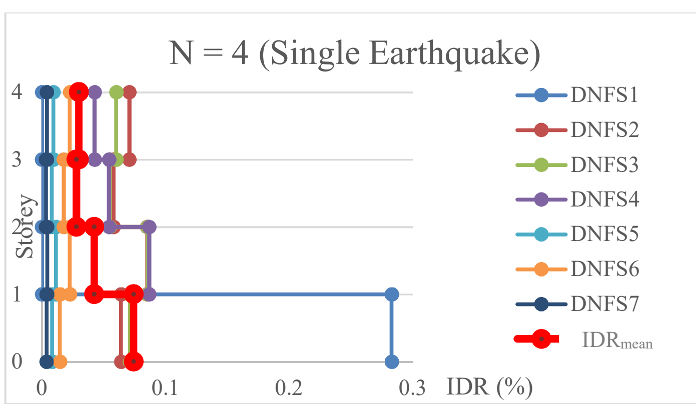

Figure D1c: Interstorey Drift Ratio (%) of single earthquake for N=4

MULTIPLE EARTHQUAKES

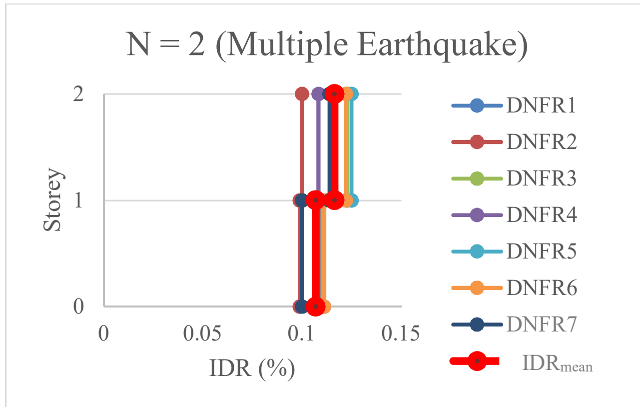

Figure D2a: Interstorey Drift Ratio (%) of multiple earthquake for N=2

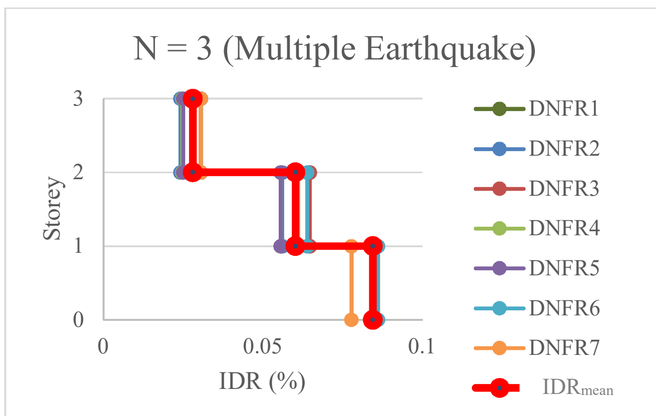

Figure D2b: Interstorey Drift Ratio (%) of multiple earthquake for N=3

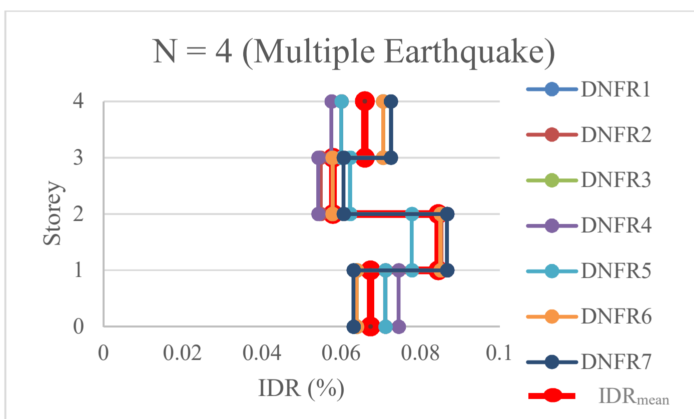

Figure D2c: Interstorey Drift Ratio (%) of multiple earthquake for N=4