EFFECT OF SOIL TYPE ON SEISMIC PERFROMANCE OF REINFORCED CONCRETE SCHOOL BUILDING

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Wassalam

ABSTRAK

Analisa ‘time history’ tidak lelurus merupakan pergerakan tanah dan tindak balas bangunan apabila dikenakan dengan gempa bumi. Amalan semasa dalam kejuruteraan gempa bumi hanya mempertimbangkan gempa bumi tunggal dalam pemodelan dan analisa. Bagaimanapun, gegaran gempa bumi akan berlaku berulang kali selepas gegaran yang pertama. Secara teknikal ianya dikenali sebagai gempa bumi berulang atau berganda. Gegaran gempa bumi berganda ini pernah berlaku pada tahun 2015 di Ranau, Sabah. Prestasi seismik bangunan dipengaruhi oleh jenis tanah dimana bangunan itu dibina. Objektif kajian ini adalah untuk mengkaji kesan gempa bumi berganda dan jenis tanah terhadap seismik bangunan konkrit sekolah bertetulang. Sebanyak 2 model bangunan sekolah yang mempunyai 2 dan 4 tingkat akan digunakan dalam kajian ini. Semua model akan direka bentuk mengikut piawaian BS8110 untuk mewakili sekolah-sekolah sedia ada di Sabah. Analisa ‘time history’ tidak lelurus telah dikendalikan pada kedua-dua model menggunakan SAP 2000. Gempa bumi tunggal dan berganda telah dipertimbangkan dalam analisa ini dengan dua jenis tanah yang berbeza. Keputusan dibincangkan dari segi anjakan sisi pada setiap tingat dan nisbah anjakan antara tingkat. Dalam kajian ini, magnitud nisbah anjakan antara tingkat semakin meningkat dari gempa bumi tunggal kepada gempa bumi berganda sekitar 5% hingga 6%. Selain itu, magnitud nisbah anjakan antara tingkat model untuk Jenis Tanah D kira-kira 14% lebih tinggi daripada Jenis Tanah B. Oleh itu, kesan gempa bumi berganda dan jenis tanah tidak boleh diabaikan dalam menganalisa dan mereka bentuk untuk membina struktur yang lebih selamat di kawasan berseismik.

ABSTRACT

The nonlinear time history analysis simulates the response of building when subjected to real earthquake ground motion. Current practice only considers single earthquake in analysis. However, the tremors of earthquake always occur repeatedly for several times after the first one. This nature can be technically called as repeated or multiple earthquake. It also occurred during 2015 in Ranau, Sabah. The seismic performance of a building also influences by the soil type. The objective of this work is to study the effect of multiple earthquake and soil types on seismic performance of reinforced concrete school building. A total of two models of school building which have 2 and 4 storeys will be used in this work. The building is assumed to be located in Sabah. All models will be design based on BS8110 to represent the current RC school building. The nonlinear time history analysis has been conducted on both models using SAP2000. The single and multiple earthquakes has been considered in the analysis with two types of soil. The results are discussed in term of the lateral displacement at each storey and the interstorey drift ratio. In this study, the magnitude of interstorey drift ratio is increasing from the single earthquakes to the repeated earthquakes around 5.0% to 6.0%. Besides, the magnitude of interstorey drift ratio of models on Soil Type D which is around 14% higher than Soil Type B. Therefore, the effect of repeated earthquake and the soil type cannot be neglected for analysis and design in order to build safer structure in seismic region.

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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
T_1	Fundamental Period
T_B	lower limit of the period of the constant spectral acceleration branch beginning of the constant
T_D	displacement response range of the spectrum
Tc	Upper Limit
S	soil factor
$S_d(T)$	design spectrum
β	lower bound factor for the horizontal design spectrum (0.2)
W	Weight
N	Number of Storey
a_g	Peak ground acceleration
a_{gR}	Reference peak ground acceleration
γ_1	Important factor
Δ_y	Yield displacement

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
NTHA	Nonlinear Time History Analysis
IDR	Interstorey Drift Ratio
SDOF	Single Degree of Freedom
UBC	Uniform Building Code
USGS	U.S Geology Survey
BNFS	Near Filed Single Earthquake on Soil Type B
BNFR	Near Filed Repeated Earthquake on Soil Type B
DNFS	Near Filed Single Earthquake on Soil Type D
DNFR	Near Filed Repeated Earthquake on Soil Type D

CHAPTER 1

INTRODUCTION

1.1 Introduction

Earthquakes known as the shaking motion of earth as the result of a sudden release of energy in the earth crust that will creates seismic waves. These are related to motion force acting within the earth crust. Plate boundaries will reserve energy due to friction between two plate faces, and earthquake will occur once the energy released when the friction are loss. Earthquake causes the movement and ground shaking then consequently causes structural building to be unstable and undergo displacement where it will be shift quickly from its original position due to the sudden seismic force. Earthquake is a movement of the surface of earth due to internal energy from the core of earth at a sudden that may cause the building to collapse and the death of thousands of people. The main cause of earthquake is the orogenic movements such as mountain building and valley farming, subduction and plate convection followed by geothermal and mechanical disturbances during volcanic activities and land erosion.

It is important to highlight that earthquakes do not need to be in large magnitude to produce severe damage. This is because the degree of damage depends not only on the physical size of an earthquake but also on other factors such as where and when an earthquake occurred, the population density in the area concerned and secondary events such as fire and also type of soil of the area.

Malaysia is situated on the southern edge of the Eurasian Plate as shown in Figure 1.1. It is close to the most two seismically active plate boundaries, the inter-plate boundary between the Indo-Australian and Eurasian Plates on the west and the inter-plate boundary between Eurasian and Philippine Plates on the east. Large earthquakes in and around these boundaries could extend and have extended to Malaysia. East Malaysia, beside of affected by large earthquakes located over Southern Philippines and in the Straits of Macassar, Sulu Sea and Celebes Sea, these two states also have experienced earthquakes of local origin. Several possible active faults has been delineated and local earthquakes in East Malaysia appear to be related to some of them.

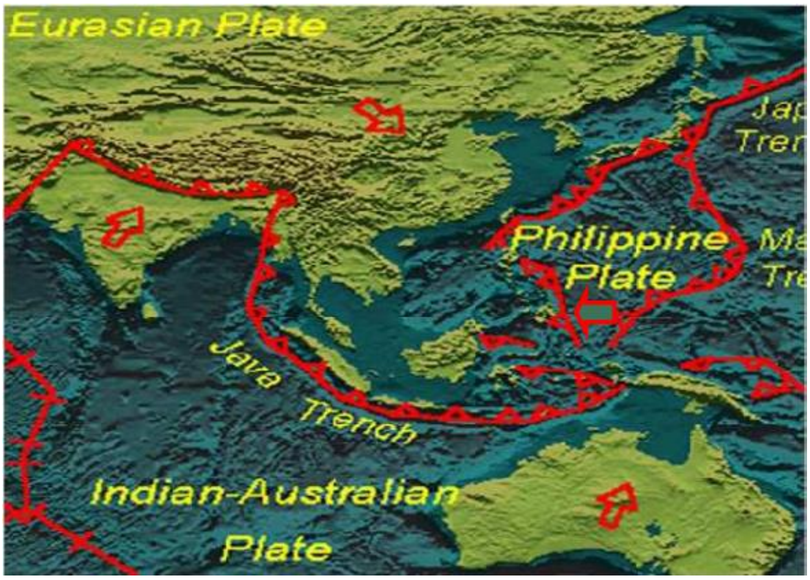


Figure 1.1 Major tectonic plates around Malaysia (Natawidjaja, D., 2001)

Figure 1.2 show the earthquake hazard zonation in Malaysia which been divided into five zone and each area has their own color that represent the level of seismicity in Malaysia. As mentioned before, Sabah is one of the high level of seismicity that reach to the level VI to VIII which it is worth noting that an earthquake of scale VIII can cause human injuries and property damages (MOSTI, 2009).

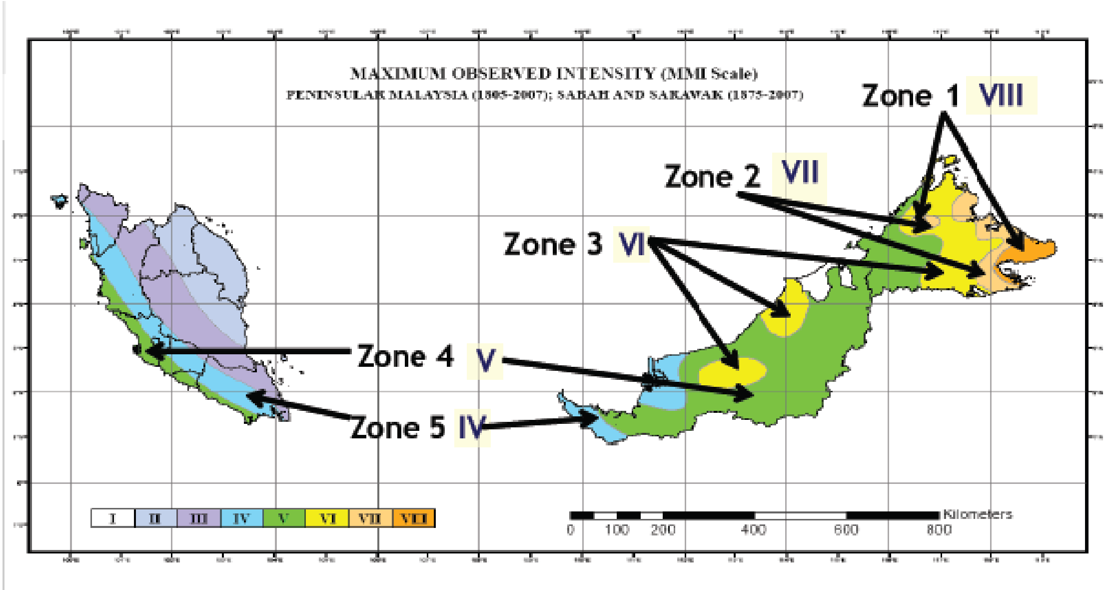


Figure 1.2 Earthquake hazard zonation in Malaysia

Malaysia has been categorized as belonging to the low seismicity group. Consequently, earthquake resistant design has not been given much emphasis until a decade ago when the Malaysian lawmakers (or Members of Parliament) were briefed by the Meteorological Department (MMD), in 2002, on the distant shock waves of the 2001 Gujarat earthquake, which travelled 600 km from its epicenter to rock and cause devastations to many cities in India (Bendick et al, 2001).

On December 26, 2004, the giant earthquake happened in Sumatran region that is closest seismic region to Malaysia shows that the seismic activity in Malaysia increased. The Sumatra earthquake with the magnitude of 7.2 that occurred on May 9, 2010 was also felt in several places in Peninsular Malaysia, even though Malaysia is not in a high seismic zone but it is surrounded by the area that are in high seismic country. That is the reason why Malaysia can feel the vibrations as well. The effect of earthquake still felt in several areas in Peninsular Malaysia even though Malaysia is not exactly located in the seismic region. It is because the seismic is being transfer through the same type of soil. With different type of soil, the level of seismicity is different.

On past few years, Malaysia was only having a small earthquake in some place which does not has much effect on the building. However, Malaysia is affected by the earthquake from another country in seismically active plate boundaries from Indonesia and Philippines. On 2015, Malaysia had face the biggest damage due to earthquake in Ranau, Sabah with magnitude of 6.0 Richter scale followed by smaller magnitude for several times. The earthquakes in Sabah are reportedly occurring due to plate convection which Sabah before was away from the boundary.

Having been affected by both local and distant ground motions, Malaysia has come to realize that seismic hazard in the country is real and has the potential to threaten the public safety and welfare, and may cause damages to properties. Such concern is attributed to the fact that less than one percent of buildings in Malaysia are seismic resistant (Ade Faisal et al, 2013).

1.2 Problem Statement

In a real earthquake event, the first tremor is always followed by other tremors. This is the nature of earthquake and may occur just a few hours after the first one, and may occur continuously to a few days. In technical views, it can be called as repeated earthquake or multi event earthquake (Hatzigeorgiou and Beskos, 2009). Table 1.1 presents the example of repeated earthquake occurred around the world.

During a great earthquake event, buildings are imposed to the action of earthquake load more than one time. The buildings may experience minor to moderate damage after being hit by the first tremor resulting in stiffness and strength degradation of the global system. For this situation, any rehabilitation action is impractical due to time constraint. Then, if the not yet repaired buildings being subjected to the following tremors, the buildings are expected to experience worse damage that lead to collapse. Current provisions in earthquake engineering such as the Eurocode 8 (2004) and FEMA 368 (2000) only suggest to considering single earthquake in analyses. In either designing the new building or evaluating the existing one, this recommendation had been practiced for years. However, it had been analytically proved that considering repeated earthquake phenomena in analysis requires an increase in strength with respect to single earthquake. Therefore, the traditional seismic design procedure, which is based on single earthquake, should be generally reconsidered.

Table 1.1 Example of repeated earthquake events around the world [Malaysian Meteorological Department, 2016]

No	Date	Time	Mw	Latitude	Longtitude	Region
1	23/07/2013 23/07/2013 23/07/2013	09:14 am 12:07 pm 12:44 pm	3.6 4.2 3.0	6.8°U 6.8°U 6.8°U	117.1°T 117.1°T 117.1°T	Kudat, Sabah
2	04/02/2014 04/02/2014 04/02/2014	04:37 am 06:36 am 07:54 am	5.0 5.7 5.0	7.5°S 7.5°S 7.5°S	128.2°T 128.2°T 128.2°T	Banda Sea
3	17/09/2015 17/09/2015 17/09/2015	06:55 am 07:18 am 11:55 am	7.9 6.4 6.3	31.4°S 31.4°S 31.4°S	71.5°B 71.5°B 71.5°B	Chile
4	21/12/2015 22/12/2015 22/12/2015	02:47 am 08:01 am 08:18 am	6.2 4.2 4.6	3.5°U 3.5°U 3.5°U	117.6°T 117.6°T 117.6°T	Tarakan, Kalimantan
5	04/09/2016 11/09/2016 16/09/2016	10:38 am 12:29 am 03:50 pm	5.7 5.0 5.3	5.6°U 5.6°U 5.6°U	125.9°T 125.9°T 125.9°T	Mindanao, Filipina

As mention in previous section, majority buildings in Malaysia had been designed without consideration of seismic load. For reinforced concrete (RC) buildings, the designed had been conducted by referring to BS8110 (1997), where seismic provision are not provided. Therefore, it is important to evaluate the existing RC buildings in Malaysia when subjected to single and repeated earthquakes. In this study, RC school buildings had been used as models because such structure is important and must survive during the earthquake.

1.3 Objectives

Objective of this study are:

i. To investigate the seismic performance of RC school building in Sabah, Malaysia when it is subjected to single and repeated earthquake.

ii. To study the effect of soil type on seismic performance of RC school building.

1.4 Scope of Work

This study covered and focused in the following aspect:

i. Two generic model of RC school buildings which have 2 and 4 storey designed based on BS8110 (1997) by using ESTEEM software.

ii. Ground motion considered as near-field earthquake (NFE).

iii. 7 number of ground motion with 2 type of combination for both single and repeated earthquake had been considered in nonlinear time history analysis (NTHA) on all models.

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

   b. Motion 2: Repeated ground motion (foreshock - mainshock - aftershock).

iv. The seismic performance had been evaluated based on the interstorey drift ratio (IDR).

v. Two type soil had been considered namely as Soil Type B (soft rock) and Soil Type D (soft soil) as referred to Eurocode 8 (2004).

vi. The reference peak ground acceleration, a_{gR} = 0.12g had been used for scaling of ground motion to represent the seismicity in Sabah.

1.5 Importance of the Study

The repeated earthquake was ignored in the current seismic code for analysis and design of buildings. However, earthquake phenomenon does not occur in single event, but earthquake is a repeated phenomenon. There could be more than two tremors after the first tremor hits the ground. Due to the action of first tremor, buildings may experience minor to moderate damages which resulting in stiffness and strength degradation. The not yet repaired building are exposed to experience greater damages which may lead to collapse when subjected to main shock and after shock. So, it is important to consider the action of repeated earthquake to evaluate and designing the structural performance of RC school building.

In Malaysia, when there is natural disaster occur, school building will be the main shelter for community to stay until the disaster dwindle. Thus, it is very important to make sure the designed RC school building in future can sustain the load or force of the repeated earthquake, which means that the building structure still can survive during the earthquake event occur.

Besides considering the repeated earthquake, type of soil where the structure will be built also should be taken as one of the factor in consideration. There are four type of soil namely as Soil Type A, B, C and D whereas it is hard rock, soft rock, stiff soil and soft soil, respectively. This study are focusing on Soil Type B and Soil Type D only. When the structure is built on the soft soil, the displacement of RC structure is high when subjected to repeated earthquake and cause greater structural damage than the single earthquake (Zhai et al, 2015).

Number of people perished in Malaysia due to the natural disaster is very high nowadays. Parties involved should play their role to make sure these problems can be reduced by considering the type of soil and repeated earthquake in designing the RC school building in Malaysia.

CHAPTER 2

LITERATURE RIVIEW

2.1 Introduction

Earthquake causes ground shaking and movement and consequently causes a structural building undergo displacement where it will be shifted in a short time from its original position due to the sudden force. Frequently a rumbling is heard several seconds before the shaking begins, and within a few seconds the initial tremors have grown into violent shaking while the other times a quake strikes like an instantaneous pulse. Generally, earthquake cause significant damage and due to the effect, the frequency of an earthquake has often been a subject in earthquake engineering.

2.2 Earthquake Hazard in Malaysia

Since 2005, the government of Malaysia has taken various efforts, through the Ministry of Science, Technology and Innovation (MOSTI), to access and address the risk associated with potential earthquake events. Research on reduction of earthquake risk in Malaysia started immediately, and some important publications are macro zonation contour maps based on peak ground acceleration (PGA) at 10% and 2% probabilities of exceedance in 50 years for bedrock of Malaysia (Adnan et al, 2006); and the assessment on the vulnerability of public buildings (Adnan et al, 2006). The Public Work Department of Malaysia (PWD) has also worked closely with academicians of local universities to establish suitable seismic design forces for use in the design of buildings.

Peninsular Malaysia has experienced weak local earthquakes and been jolted by distant earthquakes from Sumatra, East Malaysia has recorded moderate scale tremors of magnitudes between 3.6 and 6.5 between 1984 and 2007. Since 1897, the state of Sabah has recorded the highest number of ground motions country i.e. 77 earthquake events, most of which are local origin, believed to be contributed by several active faults. The maximum intensity reported was VII on the MMI scale. It is worth noting that an earthquake of scale VII can cause human injuries and property damages. The seismic activities, within Malaysia and around its region for the past 35 years, recorded between 1973 and 2008 are illustrated in Figure 2.1. A magnified pictorial of the seismicity of Sabah is as shown in Figure 2.2.

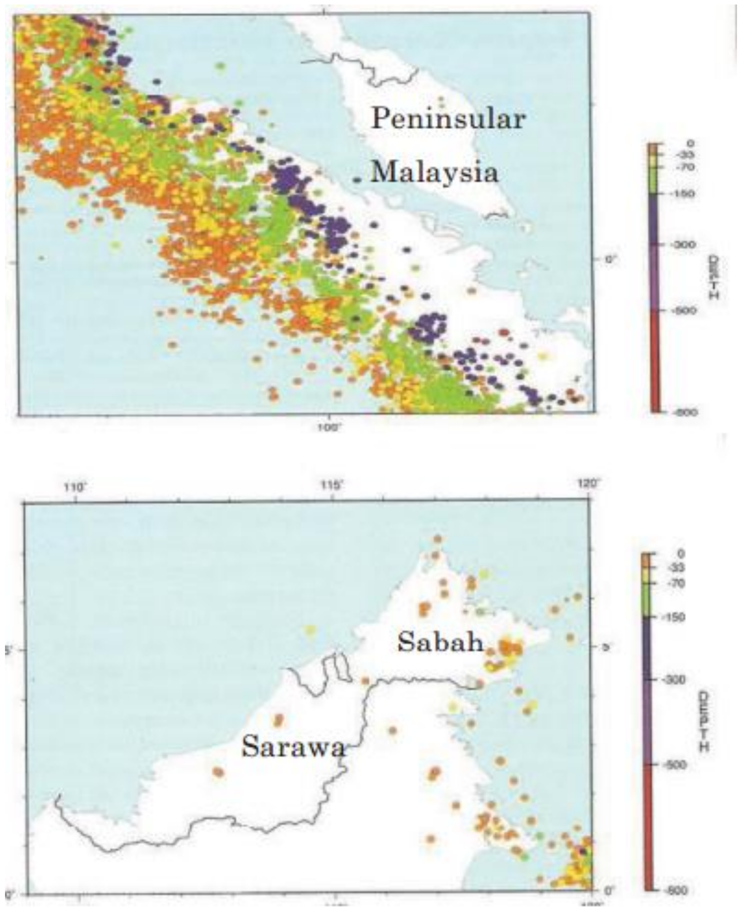


Figure 2.1 Records of earthquake epicenter in Malaysia and neighboring countries between 1973 and 2008 (adopted from USGS website)

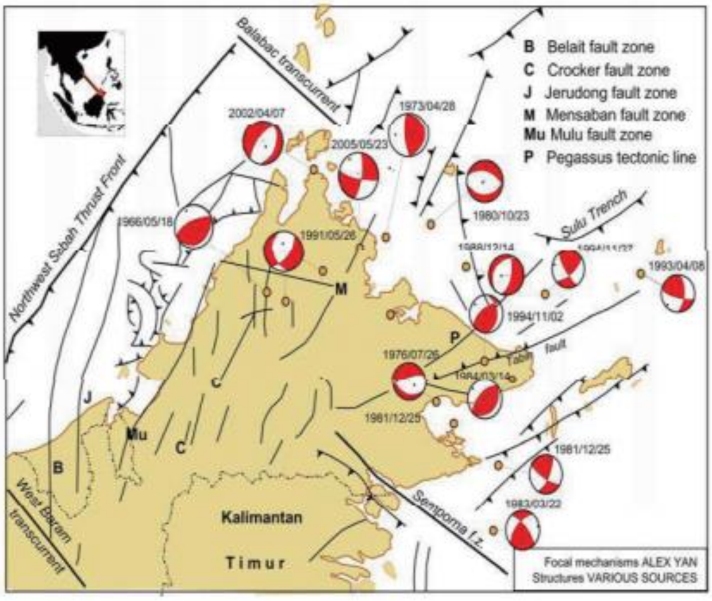


Figure 2.2 Focal mechanism of earthquake in Sabah for the period of 1976 to 2006 (MOSTI, 2009)

Nik Azizan (2010) reported 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) of earthquake in Sumatra. 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 2.3.

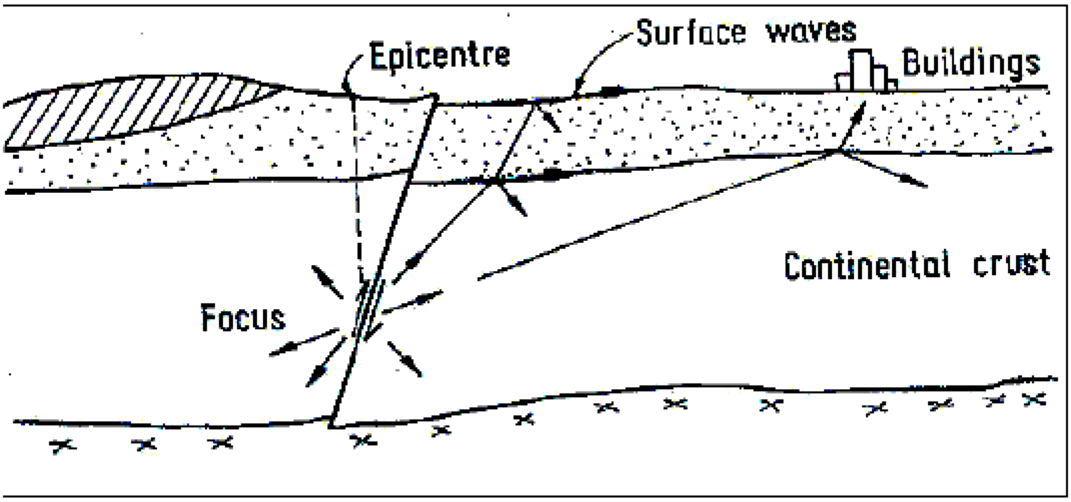


Figure 2.3 Mechanism of far-field effects of earthquakes (Balendra and Li, 2008).

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 et al., 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.

2.3 Repeated Earthquake Phenomenon

The basic cause of most earthquake by the understanding of scientist is called as dynamic process as known as continental drift of tectonic plate movement. Earthquake happened without warning and so quickly devastated their communities. Some animals, such as fish and insects have sense and they will react to earthquakes before they are felt by human. Low-intensity earthquake may be experienced as a gentle shock or a small vibration. During the intense shaking people cannot walk steadily. It will destroy the basic necessities of life, demolishing shelter, ruining food and water supplies and also disrupting people livelihoods.

It is difficult to predict the frequency of the earthquake hits the ground and may occur repeatedly (Ellen, 2000). This phenomenon will give a negative impact to the building in term of building performances. Repeated ground motion studies have been done recently but they ignored the influence of repeated earthquake in the code. The first wave of earthquake will hit the building after the ground shaking and certain displacement will occur. Permanent displacement is obviously cumulated for any other incoming ground motion and therefore, the maximum displacement appears to be increased (Hatzigeorgiou, 2010a).

In some cases, the damaged structure is not repaired after the first earthquake ground motion and it will completely deficient in the strength of structure at the end of the seismic sequence. This accumulation of damage is depending on the characteristic of the seismic events and on the type of hysteretic structural behavior (Amadio et al., 2003). These repeated earthquakes cause greater damage to the building rather than single earthquake. The not yet repaired buildings are exposed to experience greater damages and may leads to collapse when subjected to aftershock.

2.3.1 Foreshock, Mainshock and Aftershock

Foreshock, mainshock and aftershock are the basic sequences of the repeated earthquake phenomenon. The mainshock are the largest quake in a sequence occurring between the foreshocks and aftershocks. Smaller earthquake is foreshock that come before the larger quake but not all mainshock have foreshock.

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

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

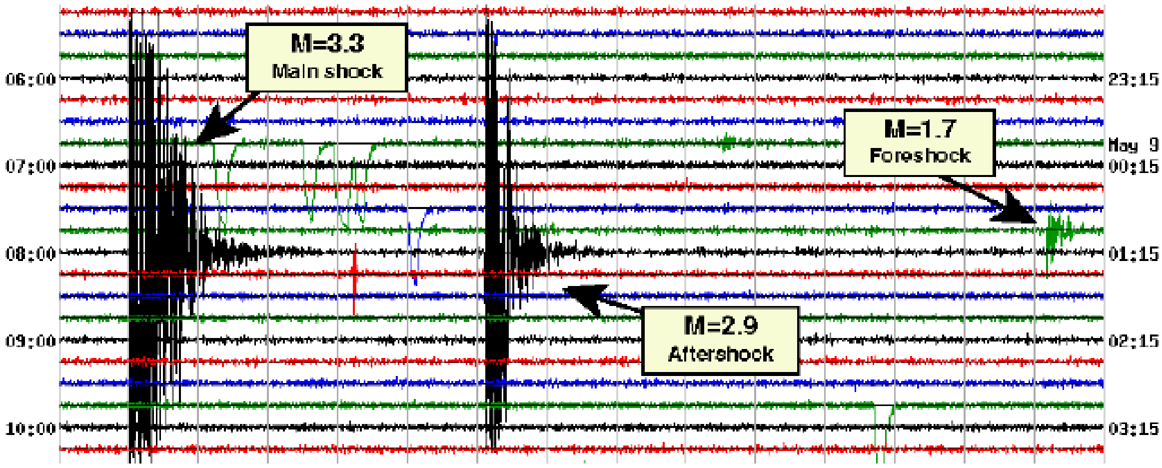


Figure 2.4 Comparison between foreshocks, main shocks and aftershocks. (USGS, 2011).

In Hubpages (2011), it state that the bigger earthquakes have more and larger aftershocks. There are could be many aftershocks within the first hour, a day, or maybe a week, month even a year of the earthquake and aftershocks decrease proportionately to the time since the mainshock happened. The bigger the mainshock, the bigger aftershock will be. In Figure 2.4 shows mainshock have larger magnitude compared to foreshock and aftershock. While the aftershock is higher compared to the foreshock, as mentioned by Hubpages (2011), the aftershock will be bigger when the main shock is high. The difference in magnitude between the mainshock 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. Aftershock of all magnitudes decrease at the same rate, but because the large aftershock are already less frequent, the decay can be noticed more quickly. Large aftershock can occur months or even years after the main shock.

2.3.2 Effect of Seismic Sequence on Low Rise Building (School building)

Malaysia is located within the low seismicity zone, and the design for earthquake ground motion is often regarded as uneconomical, unsuitable or too complex for low rise buildings. 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.

According to Tsai et al. (2000), many low rise reinforced concrete (RC) building have suffered moderate to severe damage of structural and non-structural components in earthquakes. Furthermore, it is also related to the weakness in design and construction management.

2.4 Performance Level of Buildings During Earthquake

Seismic response in term of maximum interstorey drift ratio (IDR) along the height of the structure frequently used as the engineering demand parameter (EDP) when the building was subjected to single earthquake records. A limit of 4% maximum IDR as stated in FEMA 356 (2000) need to be satisfied by RC frame building to avoid severe damage and collapse of the structure.

Krawinkler (2000) stated that the Performance-Based Earthquake Engineering (PBEE) concept include the process of design, evaluation, construction, monitoring, and maintenance of engineered facilities whose responds in various needs and objectives under common and extreme loads. This concept is based on the idea that the performance f the structures can be predicted and evaluated. Thus, the design engineer and client jointly can make better and informed decisions based on building life-cycle considerations along the construction costs.

According to FEMA 356 (2000), the structural performance levels can be classified into operational level, OL immediate occupancy level, IO life safety level, LS and collapse prevention level, CP. The target building performance levels implemented in FEMA 356 (2000) considering the PBEE concept is presented in Table 2.2.

Table 2.2 Damage Control and building performance levels (Table C1-2: FEMA 356, 2000)

	Target Building Performance Levels			
	Collapse Prevention (CP)	Life Safety (LS)	Immediate Occupancy (IO)	Operational (OL)
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse.	Some residual strength and stiffness left in all stories. Gravity-load bearing elements function. No out-of plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.
Non-structural Components	Extensive damage	Falling hazards mitigated but many architectural, mechanical, and electrical systems are damaged.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.	Negligible damage occurs. Power and other utilities are available, possibly from standby sources.

In operational level, OL the structure retains its original stiffness and strength where the negligible damage occurs to non-structural components. The risk of life-threatening is very low. At immediate occupancy level, IO it is expected that very limited structural damages occur due to earthquake loads where the buildings retain nearly all of their pre-earthquake strength and stiffness. Thus, the risk of life-threatening injury is very low and minor structural repairs may be appropriate for re-occupancy.

Significant damage to some structural elements is expected in the life safety level, LS but this has not resulted in large falling debris hazards. Injuries may occur during earthquake but the overall risk of life-threatening is expected to be low. The damage is still possible to be repaired but for economic reason this may not be practical. For re-occupancy, it would be safer to implement structural repairs or install temporary bracing even the damaged structure is not an imminent collapse risk.

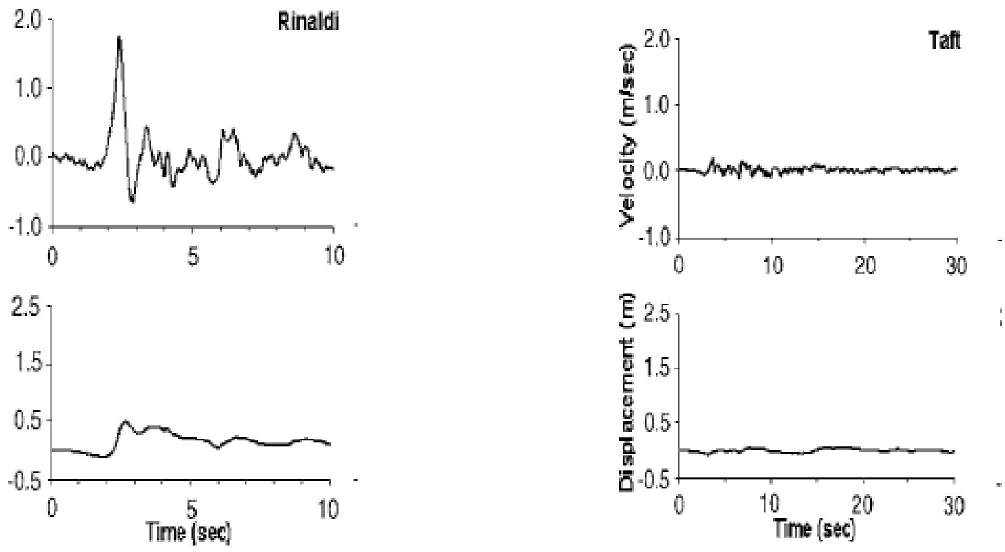
Collapse prevention level, CP means that the building is on the verge of partial or total collapse due to earthquake loads. At this level, severe damage has occurred to structure including significant stiffness and strength degradation, large permanent lateral deformation of the structure even the degradation in vertical load carrying capacity. The risk of injury due to falling hazards from structural debris is high. Technically, the structure may not be practical to repair. Hence, at collapse prevention level, CP the structure is not safe for re-occupancy. The aftershock activity could induce collapse.

2.5 Ground Motion

Ground motion is a movement of earth's surface with the passage of earthquake waves, which radiate energy that had been stored in stressed rocks, and were released when a fault broke and the rocks slipped to relieve the pent-up stress. The strength of ground shaking is measured in the velocity of ground motion, the acceleration of ground motion, the frequency content of the shaking and how long the shaking continues (the duration). When assessing the potential shaking hazard at a given site, how frequently strong motion is expected to re-occur there is another critical factor to be considered.

Interestingly, the characteristics of waves produced by an earthquake rupture are also strongly influenced by the fault rupture orientation, its depth, and the details of how the slip spread across the ruptured fault patch. A rupture may also start at, or propagate into, strong patches along a fault plane. If the strong patch fails, it may slip more than the surrounding fault and radiate more strongly than the average seismic energy.

The different of the far field earthquake (FFE) and near field earthquake (NFE) is due to the velocity pulse. According to Bayraktar et al, (2009), NFE are characterized by 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 that 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 epicenter of the earthquake is within 80 km (Razak, 2010). Figure 2.5 shows the comparison of FFE and NFE in terms of velocities and displacement time histories.



(a) (b)

Figure 2.5 Typical velocity and displacement time histories (a) Far-Field (b) Near-Field Earthquakes (Kalkan and Kunnath, 2006)

The existence of high velocity pulse in NFE with forward directivity effect is believed to induce severe damage compared to ordinary earthquake that not contain pulse characteristic (Alavi and Krawinkler, 2004; Kalkan and Kunnath, 2006; and Baker, 2007). Even have the same PGA, the forward directivity effect in NFE with pulse characteristic tends to require larger ductility demand compared to the ordinary earthquake (Sehhati et. al, 2011). In addition, Kalkan and Kunnath (2006) concluded that as the pulse period, T_P approaches the fundamental period of the building, T_1 the structural demands were clearly amplified.

2.6 Method of Analysis

Earthquake engineering has become one of the most important fields in engineering world. There are many methods in evaluate building performance during earthquake event. According to Eurocode 8 (2004), the following methods are usually used in the assessment of the building performance:

1. Static analysis (technically known as “pushover” analysis), using equivalent increasingly lateral force acting on the structure.
2. Dynamic (time-history or response-history) analysis, either modal response spectrum analysis or time history analysis with numerical integration using earthquake records.

2.6.1 Nonlinear Time History Analysis

The Nonlinear Time History Analysis (NTHA) is the one of the dynamic analysis. NTHA is employed when the entire time history of the elastic response is required during the ground motion of interest. Under a specific ground motion, the dynamic response of the structure is computed at each time increment and the structure is modelled with due considerations given to stiffness and mass distribution and others factors. Then the model tested using a pairs of ground motion time history components which compatible with design response spectra in the building codes.

The requirement to conduct time history analysis for an ensemble of ground motion records that represent magnitudes, fault distances, and source mechanisms that are consistent with those of the design earthquakes used to generate design response spectra. The ground motions used in this analysis either the previously recorded ground motions, scaled to match the design response. Besides that, the artificially generated records with similar properties also can be used in the analysis. Depending on the number of ground motion records considered, the average of design parameter obtained from different analyses is used in design.

Saatcioglu and Humar (2003) stated that nonlinear analysis allows for flexural yielding (or other inelastic actions) and accounts for subsequent changes in strength and stiffness. Hysteretic behaviour under cyclic loading is evaluated. Hysteretic behaviour of the elements that make up the structure is the most important in differentiate between linear and NTHA. This behaviour is incorporated into analysis computer software through hysteric models.

The hysteretic behaviour depends on the characteristics of structural materials, design details, and a large number of design parameters, as well as the history of loading. The maximum displacement experienced by the structure and maximum ductility demands in the member can be determined using NHTA. Saatcioglu and Humar (2003) specified if the ductility demands are less than the ductility capacities and the deflections are within acceptable limits, the design is satisfactory.

From Figure 2.6, 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.

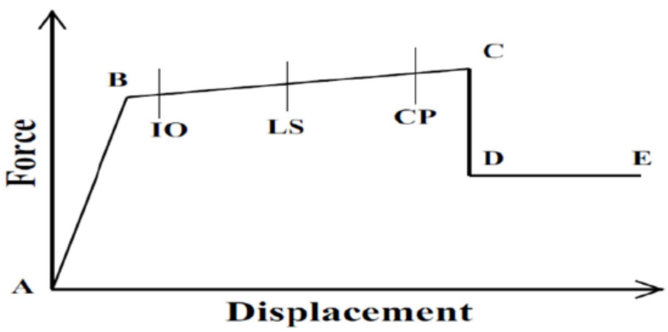


Figure 2.6 Strength and deformation points (FEMA 356, 2000)

2.7 Effect of Repeated Earthquakes on Structural Performance

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

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.

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.

Among the earliest work to study the effect of repeated earthquake on structural performance was conducted by Amadio et al. (2003). SDOF model was used in dynamic analysis considering one real and two artificial repeated earthquake records. In artificial earthquakes, a gap of 40 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 repeated earthquake is very similar to the response of the same structure under single earthquake excitation. However, the repeated 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 repeated earthquake phenomenon.

Hatzigeorgiou and Beskos (2009) runs million NTHA on the elasto-plastic SDOF system with strain hardening and empirically introduced an expression to estimate the inelastic displacement ratio (defined as the ratio of maximum inelastic displacement and maximum elastic displacement) on the flexural-based RC and steel structures based on period of vibration, force reduction factor, site conditions, post-yield stiffness, and damping. It concludes that repeated earthquakes require increased displacement demands in comparison with single seismic events as in design earthquake. They find the inelastic displacement ratio for all SDOF systems built at all soil types generally appear to be increased 2 times or more with respect to that obtained for the corresponding single earthquakes.

The effect of repeated 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 repeated 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 repeated 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 repeated earthquake excitation had been investigated by Ruiz-Garcia and Negrete-Manriquez (2011). The NTHA 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 aftershock 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

Faisal et al. (2013) have studied three-dimensional moment resisting frame under bi-directional seismic excitation. It explains that the story ductility demand of low-story reinforced concrete buildings are significantly affected by repeated earthquakes. It finds that as the force reduction factor decreases the effect of repeated earthquake is decreases. The effect could be neglected when the structures have force reduction factor of less than two. The upper level of the short structure tends to have the maximum demand when experiencing repeated earthquakes.

Adiyanto and Majid (2014) conducted NTHA on three storey reinforced concrete hospital buildings build on Soil Type D (soft soil) in Malaysia. The three storeys with two bays model was design based on behavior factor, q equal to 2.3, 3.1, 3.9, 4.7, and 5.5 respectively. The author concludes that the action of multiple earthquake on frame tend to cause higher IDR compared to action of single earthquake. The author concludes 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 repeated earthquake. The latter also caused greater structural damage than the single earthquake. Therefore, the authors strongly suggested that the repeated earthquake phenomenon cannot be neglected and the traditional response spectrum, which is based on single earthquake, should be further evaluated.

2.8 Summary

The repeated earthquake was ignored in the current seismic code and design of buildings in Malaysia. However, recently many studies had done on repeated earthquake and the effects of repeated earthquake to the building are found to have higher damages compared to single earthquakes. Therefore, it is important to extent the importance of considering repeated earthquakes to the design and evaluation of RC school buildings in Sabah, Malaysia.

CHAPTER 3

METHODOLOGY

3.1 Introduction

This chapter consists of four main sections. The first section introduces both 2D generic frame models used in this research. Two types of reinforce concrete (RC) building with 2 and 4 storey single bay moment resisting frame were selected to represent the existing school buildings. Engineering assumptions such as the storey stiffness distribution along the height of the building is also discussed in detail.

The second section in this chapter explains on how all sets of artificial repeated earthquakes from near-field earthquake (NFE) were generated for nonlinear time history analysis (NTHA) in this research. Full list of single original NFE consist of forward directivity effect can be reviewed in this section. The combination of single earthquake to generate artificial repeated earthquake also highlighted.

The performance between two type of soil namely as Soil Type B and Soil Type D on both models can be reviewed in detail in the third section of this chapter. SAP 2000 program was used for this study. Finally, the detail of NTHA considering single and repeated earthquake on both models using SAP 2000 program is discussed in the fourth section. The interstorey drift performance of the models is evaluated based on the lateral displacement of each of the storeys of the models on both type of soils when subjected to the ground motions. The overview of the methodology in this research is presented by a flowchart in Figure 3.1.

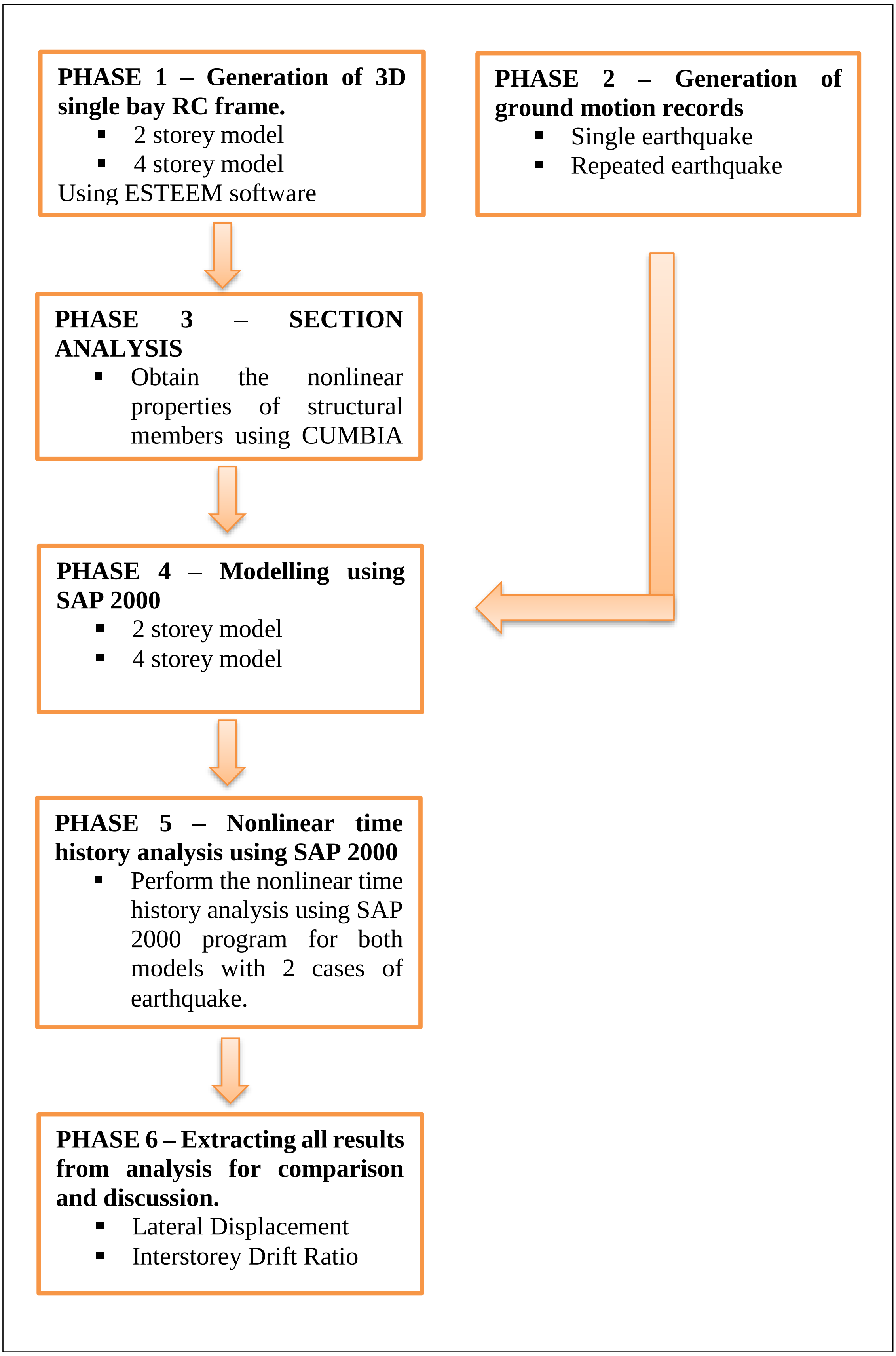


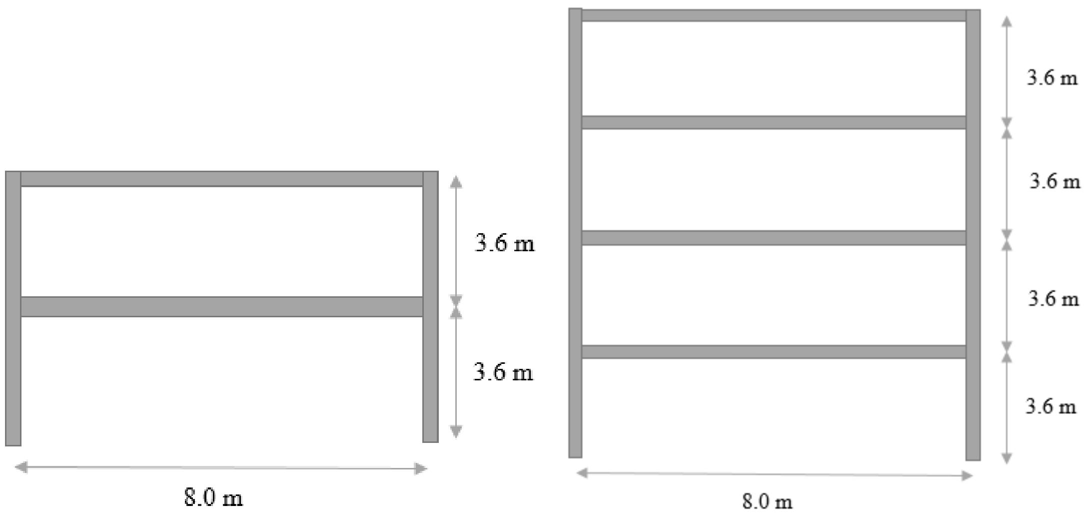
Figure 3.1 Flow chart of research methodology

3.2 2D Generic Frame Model

This section briefly discusses the RC frame building used as model in this research work. Firstly, the background of 2D generic RC frame is highlighted before followed by other sub-section such as stiffness and strength distribution of generic frame model.

3.2.1 Background of 2D Generic RC Frame

As briefly mentioned earlier, this research work used 2D multi-storey single bay system to represent the existing RC school building in Sabah. The low rise RC building were represented by 2 and 4 storey generic models. Both 2D generic frame models are in square shape. Figure 3.2 presents the elevation view of generic RC frames used in this study. The shapes of the model are rectangular in plan with 3.6 meter storey height and the beam span is 8.0 meter. Both models are regular in plan and elevation.



(a) (b)

Figure 3.2 2D generic model of (a) 2-storey and (a) 4-storey frame

Since this research neglecting the torsion effect due to its regular square shape, the mass of each floor is assigned at the center point of the corresponding floor of the structure. At the first mode and other modes where 95 percent of mass participation attained, five percent of Rayleigh damping, which is commonly used for RC building was used in this research.

In this study, both of the frames are been designed for school classroom which is classified in Eurocode 1 (2002) as Category A. Therefore, the live load, Q_k imposed on the floor and roof (accessibility for maintenance and repairing job) is equal to 2.0 kN/m^2 and 0.4kN/m^2, respectively. Proposed by Mc Kenzie (2004) and Arya (2009), weight of materials, which contributes to dead load, G_k, is shown in Table 3.1 below. In addition, as mention earlier in Chapter 1, this study considers two types of soil namely as Soil Type B (soft rock) and Soil Type D (soft soil) by referring to Eurocode 8 (2004).

Table 3.1 Weight of materials (Mc Kenzie, 2004)

Material	**Weight**	**Unit**
Concrete	24.0	kN/m^2
Finishing	1.0	kN/m^2
Water proofing	0.5	kN/m^2
Suspended ceiling	0.15	kN/m^2
Mechanical and electrical	0.3	kN/m^2
Brickwall	3.0	kN/m^2/m height

The RC design has been performed which is currently use in buildings design in Malaysia based on BS8110 (1997). The concrete compressive strength, f_{cu}=30 N/mm^2 and yield strength of steel, f_y= 460 N/mm^2. The 2 storey school building has the column size is equal to 400 mm x 400 mm and size of beam at the the roof and each floor is equal to 250 mm x 500 mm. 4 storey school building consist of the same size of all column and beam as mention in 2 storey school building. Therefore, a total of 4 frames (2 frames for Soil Type B and 2 frames for Soil Type D) has been produced as model for NTHA.

3.2.2 Fundamental Period of Vibration, T_1

In this research, the fundamental period of vibration, T_1 of a structure for both 2 and 4 storey models is determined by using the equation proposed by equation 4.6 in Eurocode 8 (2004) as follow:

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

where T_1 is the fundamental period of vibration and H is the height of the building, in meter, from the foundation of from top of a rigid basement. Since this study only focus on RC moment resisting frame system, the coefficient value of C_t in the equation (3.2) is taken as 0.075 as stated by Clause 4.3.3.2.2b(3) in Eurocode 8 (2004).

3.3 Design Response Spectrum

From equation (3.1), 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. A series of design response spectrum had been generate 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 B and 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.

$$0 \leq T \leq T_B: S_c(T) = a_g \, . S \, . \left[1 + \frac{T}{T_B} \, . (\eta \, . 2.5 - 1)\right] \tag{3.2}$$

$$T_B \leq T \leq T_C: S_c(T) = a_g \, . S \, . \eta \, . 2.5 \tag{3.3}$$

$$T_C \leq T \leq T_D: S_c(T) = a_g \, . S \, . \eta \, . 2.5 \left[\frac{T_C}{T}\right] \tag{3.4}$$

$$T_D \leq T \leq 4s: \; S_c(T) = a_g \,.\, S \,.\, \eta \,.\, 2.5 \left[\frac{T_C T_D}{T^2}\right] \tag{3.5}$$

where:

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

a_g = design ground acceleration on Type A ground ($a_g = \gamma \mathrm{L} a_{g\mathrm{R}}$)

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 = behavior factor

$S_c(T)$ = elastic response spectrum

η = damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping.

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 according to Eurocode 8 (2004). The value of each parameters in Soil Type B and Soil Type D are presented in Table 3.2 below.

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)**
B	1.20	0.15	0.50	2.00
D	1.35	0.20	0.80	2.00

3.4 Design Ground Acceleration, a_{gR}

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

$$a_g = \gamma_1 . a_{gR} \tag{3.6}$$

where;

γ_1 = importance factor

a_{gR} = reference peak ground acceleration

The value of importance factor, γ_I is depends on the importance classes of building. 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. According to Fardis et al. (2015) the recommended importance factor, γ_I is to offer better protection of life for such buildings due to its importance after disaster. Therefore, the value of importance factor, γ_I is equal to 1.2 since the school building is categorized under important class III.

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

Importance Class	**Buildings**	**Importance factor, γ_I**
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	0.8
II	Ordinary buildings, not belonging in the other categories.	1.0
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, 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

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 published by MOSTI (2009). The seismic hazard map referred in this study is shown in Figure 3.3 for Eastern Malaysia. The value of PGA is in unit gal, whereas 1 gal is equal to 0.001g. Therefore, the value of PGA for Eastern Malaysia from Figure 3.3 is lies in range of 60 gals to 120 gals which is equal to 0.06g to 0.12g. Thus, to represent the moderate seismic region in Sabah, Malaysia the reference peak ground acceleration, a_{gR} is equal to 0.12 had been used to develop the design response spectrum.

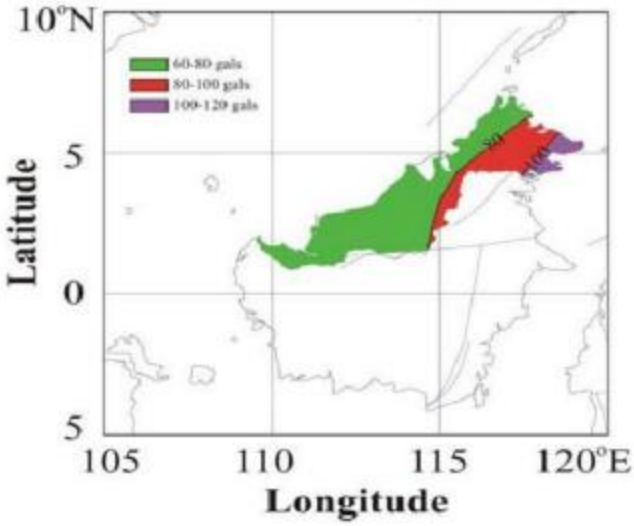


Figure 3.3 Seismic hazard map for Eastern Malaysia (Adnan et al., 2008)

3.5 Ground Motion Records

This section briefly discusses the seismic input motion used in the NTHA. The full list of single NFE used in this study can be reviewed before the process of generating the artificial repeated earthquakes is clearly highlighted.

Generally, to perform the NTHA, two set of ground motion records namely as single and multiple earthquakes had been used in this study. Each set has 7 number of ground motion records. Table 3.4 shows the list of NFE ground motion records for Soil Type B which can be downloaded from PEER database. In this study, the multiple earthquakes ground motion records had been artificially generated from random combination of single ground motion records from Table 3.4. Each multiple earthquakes ground motion record has three component namely as Foreshock, Mainshock, and Aftershock as used in previous studies (G.D. Hatzigeorgiou, 2010). The list of NFE ground motion records for Soil Type D is presented in Figure 3.5.

Table 3.4 NFE ground motion records of single earthquakes for Soil Type B

No	Event	Component	Station	PGA (g)	PGV (cm/s)	PGD (cm)	Mw
1	Palm Springs	MVH045	5071 Morongo Valley	0.218	31.4	8.51	6.0
2	Northridge	BVA195	638 Brentwood VA Hospital	0.187	23.7	5.42	6.7
3	Westmoreland	PTS315	5051 Parachute Test Site	0.155	26.6	12.97	5.8
4	Chi Chi	ALS E	ALS	0.183	39.3	10.37	7.6
5	Chi Chi	ALS N	ALS	0.163	21.9	8.64	7.6
6	Chi Chi	CHY029 N	CHY 029	0.238	35.2	29.10	7.6
7	Chi Chi	NSY N	NSY	0.128	41.8	28.72	7.6

Table 3.5 NFE ground motion records of single earthquake for Soil Type D

No	Event	Component	Station	PGA (g)	PGV (cm/s)	PGD (cm)	Mw
1	Imperial Valley	H-E03140	5057 El. Centro	0.266	46.8	18.95	6.5
2	Imperial Valley	H-E03230	Array #3 5057 El. Centro	0.221	39.9	29.29	6.5
3	Loma Prieta	A02043	Array #3 1002 APEEL2 Redwood City	0.274	53.6	12.53	6.9
4	Loma Prieta	A02123	1002 APEEL2 Redwood City	0.220	34.3	6.83	6.9
5	Chi Chi	ALS N	ALS	0.163	21.9	8.64	7.6
6	Chi Chi	CHY029 N	CHY 029	0.238	35.2	29.10	7.6
7	Chi Chi	NSY N	NSY	0.128	41.8	28.72	7.6

3.5.1 Seismic Sequence

The selected of 7 NFE single ground motion records is scaled by using Matlab program to match with the design response spectrum of Sabah as proposed by Eurocode 8 (2004). The scalling of real NFE to response spectrum is presented in the Table 3.6 below. Next, by using the same program, the single ground motion was combined randomly to stimulate the artificial repeated earthquakes. The table of combination of the single earthquakes to repeated earthquakes that consist of foreshock, mainshock and aftershock for Soil Type B are shown in Table 3.7. The similar details for Soil Type D are attached in Appendix A.

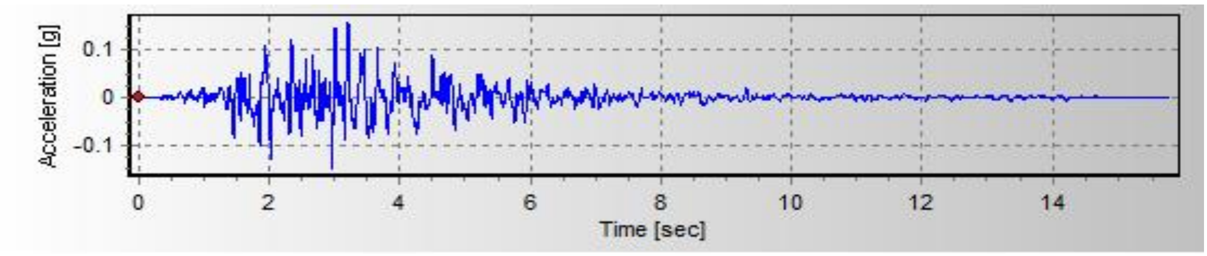
Table 3.6 Scalling of real NFE to response spectrum for Soil Type B

Fundamental period of vibration, T_1 = 0.17 sec			
No.	Sa(T_1), g		Scale Factor (b/a)
	Original Record (a)	Response Spectrum (b)	
BNFS1	0.3527	0.4320	1.2248
BNFS2	0.3395	0.4320	1.2725
BNFS3	0.1792	0.4320	2.4107
BNFS4	0.6624	0.4320	0.6522
BNFS5	0.1146	0.4320	3.7696
BNFS6	0.5541	0.4320	0.7796
BNFS7	0.2982	0.4320	1.4487

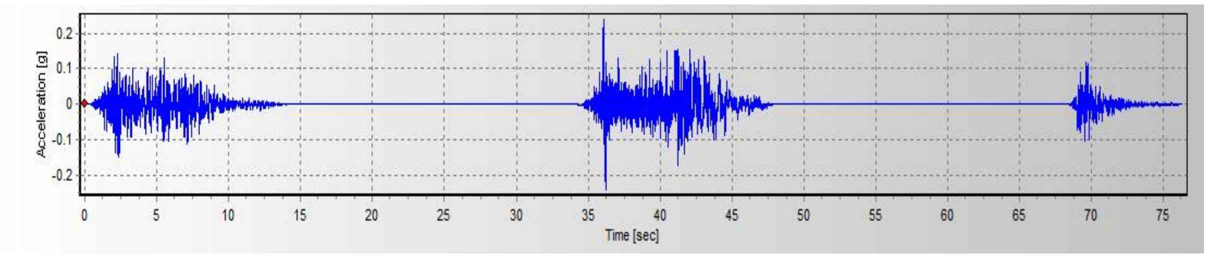
Table 3.7 Combination of single earthquakes to generate repeated earthquakes for Soil type B

NO.	FORE-SHOCK	MAIN-SHOCK	AFTER-SHOCK
BNFR1	BNF1	BNF3	BNF5
BNFR2	BNF2	BNF4	BNF6
BNFR3	BNF1	BNF4	BNF5
BNFR4	BNF3	BNF2	BNF6
BNFR5	BNF5	BNF4	BNF3
BNFR6	BNF2	BNF6	BNF1
BNFR7	BNF4	BNF2	BNF6

These ground motion records used in the analysis are divided into two types of motion. For motion 1, single ground motion which only consider the mainshock. This is called as single earthquake. Whereas the motion 2 consists of the three ground motion (foreshock- mainshock – aftershock). In Appendix A shows all, the details regarding NFE single earthquakes and repeated earthquakes for both type of soils, Soil Type B and Soil Type D that had been imposed to the all models for NTHA A time gap is also applied in between two consecutive seismic events in this case to cease the moving 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 adequate to cease the moving of any structure due to damping as suggested by Hatziegeorgiou and Liolios (2010). Figure 3.4 shows the typical profile generated ground motion for all the cases.



(a)



(b)

Figure 3.4 Typical profile of generated ground motion with 100s gaps

3.6 Section Analysis by using Cumbia

In this study, the section analyses were performed by using CUMBIA software (Montejo and Kowalsky, 2007). In order to run the NTHA, nonlinear properties of all structural elements, which is beams and columns of both frames has to be determined through simple process namely as section analysis. CUMBIA program is a set of Matlab code used to perform monotonic moment curvature analysis of reinforced concrete members. Before run the section analysis, the input in term of structural geometry and its steel reinforcement had to be assigned alongside the strength of material. 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.

To conduct these analyses, open the software and choose either beam or column for analysis. After the file is open, the file name is inserted. “n” is inserted for interaction. First thing to be done is named the file. 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 bar - 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:

f_{pc} = 30; Concrete compressive strength (MPa)

E_c = 26000; Concrete modulus of elasticity (MPa)

f_y = 460; Longitudinal steel yielding stress (MPa)

f_{yh} = 250; Transverse steel yielding stress (MPa)

E_s = 205000; Steel modulus of elasticity

f_{su} = 600; Longitudinal steel maximum stress (MPa)

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

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

Next, after complete the material properties, save the file and “debug” button is selected to run the analysis. Several number of figures will be shown after the process of analysis is complete. In Figure 3.5 and Table 3.8 presents the sample of moment-curvature curve for beam and the data from moment-curvature curve. The output from these analysis is in excel file. User SF, hinge length, ultimate moment, ultimate curvature, yield moment and yield curvature from CUMBIA are the data specified in SAP2000 for nonlinear analysis.

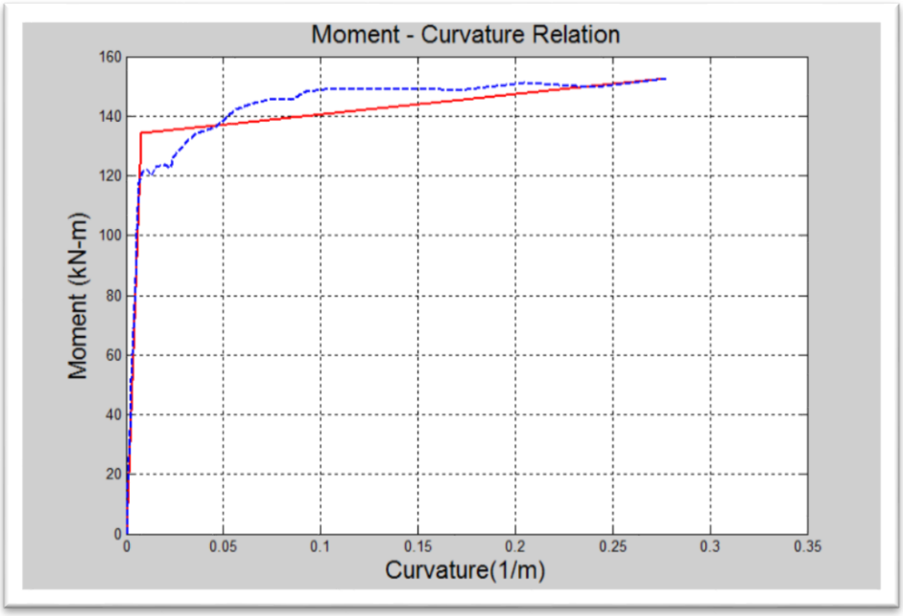


Figure 3.5 Moment-curvature curve (Cumbia)

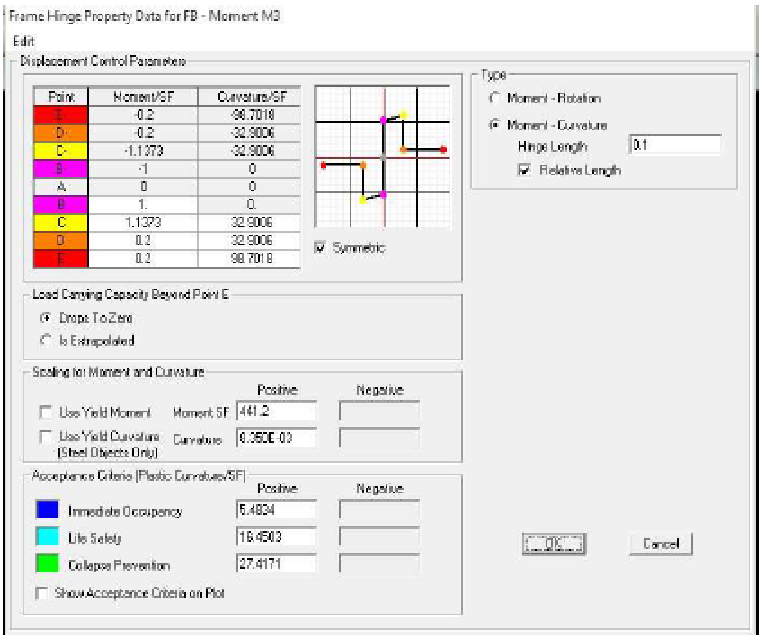
Table 3.8 Moment-curvature data Cumbia)

	CURVATURE	MOMENT
YIELD	1	1
ULTIMATE	34.56258993	1.141371032

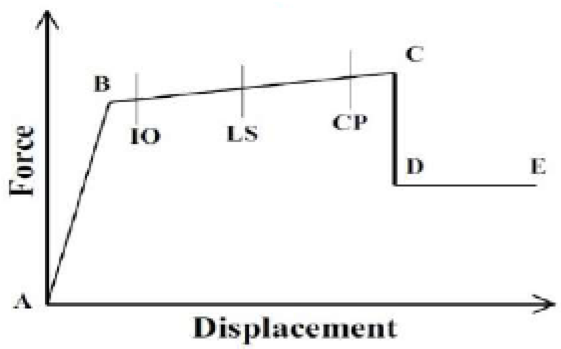
3.7 Plastic Hinge Properties at Member's End

Continue the next steps with analyzing hinges at member's ends. Moment-curvature data are used in this section and hence hinge length is assumed to be 0.5H of member section because it is dealing with RC frame (Park and Paulay, 1975). This will give a force-displacement curve with strength and deformation points as shown in Figure 3.6. Value used for SAP2000 is the point B-C-D-E. The procedure is repeated for different strength and deformation.

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. Thus, this step is repeated for all structural members of the models.



(a)



(b)

Figure 3.6 Assigning the hinge property in SAP 2000 (a) Force-Displacement Relationship and (b) Defining Frame Hinge Properties

3.8 Data and Analysis

The NTHA had been conducted on all model by using SAP 2000. The lateral displacement of every storey for each of the models were obtained when subjected to single earthquake and multiple earthquake by using NTHA on SAP2000. Thus, the value of interstorey drift ratio (IDR) of both 2 storey and 4 storey RC school building can be obtained for investigating and determine the performance of the buildings when subjected to single earthquake and repeated earthquake on two different type of soil, namely as Soil Type B and Soil Type D.

3.9 Summary of Procedure

In general, this section shows the procedure that can be summarised as follows:

1. Determine type of buildings model and design 2 and 4 storey RC school buildings models by using ESTEEM software to represent the existing buildings.
2. Perform section analysis on the structural members of the models by using CUMBIA software to determine the nonlinear properties and hinge properties of the models.
3. Produce the 2 and 4 storey RC school models by using SAP2000.

4. Select the seismic region including the two different soil type (Soil Type B and Soil Type D) in order to develop the design response spectrum.
5. Selecting seven number near field ground motion records from all over the world.
6. Calculate the fundamental period (T_1) based on the total height of the model.
7. Scaling of selected single near field ground motion records to match the designed response spectrum by using Matlab and Seismosignals software.
8. Generate seven artificial repeated earthquakes from randomly selected single near field ground motion record by using Matlab.
9. Run the NTHA of the RC school models subjected to single and repeated earthquake by using SAP2000
10. Obtain maximum lateral displacement of the model when subjected to single and repeated earthquake.
11. Calculating the IDR for every storey of the buildings when subjected to single earthquake and repeated earthquake.
12. Repeating from step 9 for the next models.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Introduction

This chapter presents the results and discussion based on analyses performed. In order to estimate the nonlinear behavior of structural system against the earthquake, nonlinear time history analysis (NTHA) is one of the mode to get the results. The NTHA on two RC frames (2 and 4 storey models) that has been analysed using SAP2000 program. The ground motion was divided into two, which are single earthquakes and repeated earthquakes. Two types of soil had been considered namely as Soil Type B and Soil Type D as discussed in Chapter 3.

4.2 Nonlinear Time History Analysis

4.2.1 Lateral Displacement

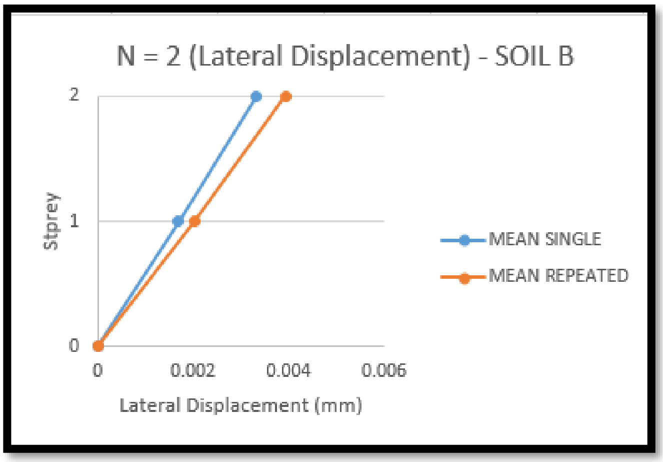
From the NTHA, the result obtained was maximum lateral displacement at each joint of the model for 2 and 4 storey models when subjected to 7 number of ground motion with 2 cases which is single earthquakes and repeated earthquakes. The results of maximum lateral displacement (in meter) after the 7 single earthquakes been imposed to the 2 and 4 storey model with two different type of soil namely as Soil Type B and Soil Type D are attached in Appendix B. Appendix C presents the result of 7 repeated earthquake that has been imposed to the 2 and 4 storey models with the same two type of soils.

As referred to Appendix B, for the 2 storey model (N=2) on Soil Type B, the highest value of lateral displacement is caused by BNFS2 which contribute 0.00384 m displacement at the top floor while the smallest lateral displacement was caused by BNFS7 which is 0.00256 m. Thus, to ensure the precise and accuracy of this study, mean value of the lateral displacement has to be taken from the 7 different ground motion records. The distribution of lateral displacement and mean lateral displacement when subjected to single earthquake and multiple earthquake is presented in Appendix D and Appendix E, respectively.

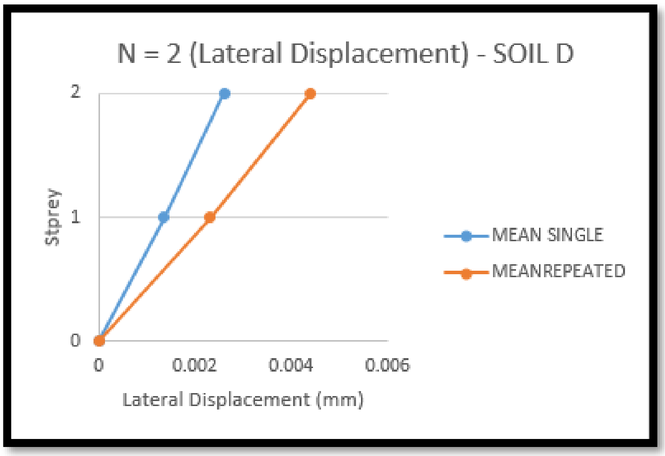
Figure 4.1 presents the mean lateral displacement (in meter) of 2 storey and 4 storey school building on 2 type of soil which are Soil Type B and Soil Type D. In Figure 4.1(a), it can be seen that the maximum mean lateral displacement caused by repeated earthquake on Soil Type B is equal to 0.0039m at the top level. While, the maximum mean lateral displacement caused by single earthquake is equal to 0.0033m at the top level. Thus, the mean lateral displacement caused by repeated earthquake is relatively 15.38% higher compared to single earthquakes.

Similar to Soil Type B, Figure 4.1(b) presents the mean lateral displacement of 2 storey school building on Soil Type D. The value of maximum mean lateral displacement for single earthquake is equal to 0.00260m whereas the value of maximum mean lateral displacement for repeated earthquake increasing 41.01% which is equal to 0.00441m. It is clearly proved that repeated earthquake is always higher than single earthquake.

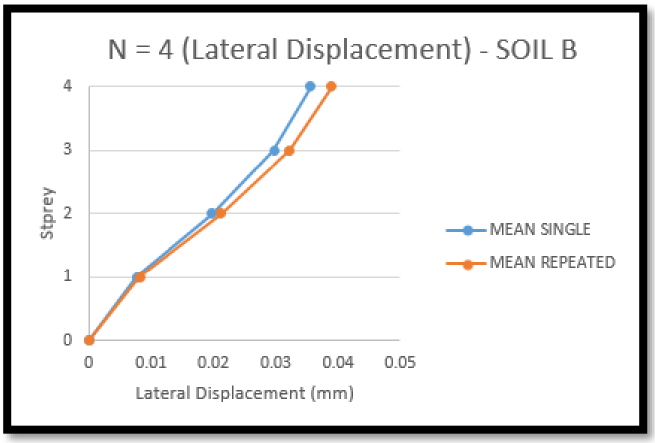
Next, Figure 4.1(c) and Figure 4.1(d) present the maximum mean lateral displacement of 4 storey school building when subjected to single and repeated earthquake on Soil Type B and Soil Type D, respectively. The value of maximum mean lateral displacement for single earthquake on Soil Type D is 15.25% higher than Soil Type B with difference of 0.00064m. While for repeated earthquake, the maximum value of mean lateral displacement on Soil Type B is equal to 0.03888m while in Soil Type D is equal to 0.04364m. The difference of these two value are 10.90% and it is prove that the lateral displacement on Soil Type D is always higher than Soil Type B.



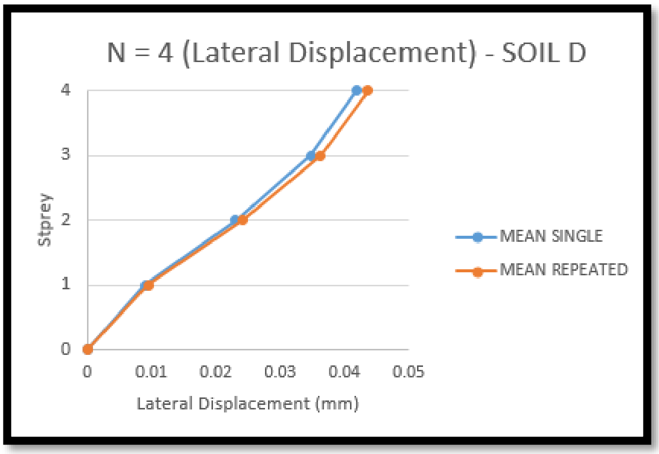
(a) Mean lateral displacement of single and repeated earthquakes for N=2 on Soil Type B



(b) Mean lateral displacement of single and repeated earthquakes for N=2 on Soil Type D



(c) Mean lateral displacement of single and repeated earthquakes for N=4 on Soil Type B



(d) Mean lateral displacement of single and repeated earthquakes for N=4 on Soil Type D

Figure 4.1 Mean lateral displacement (mm)

4.2.2 Interstorey Drift Ratio

Appendix F and Appendix G present the results of the maximum percentage of interstorey drift ratio (IDR) of every models for single and repeated earthquakes, respectively on both type of soils. The result of this study will be discussed in term of

IDR (%) value for each storey that has been imposed with the single earthquakes and repeated earthquakes for both type of soils, namely Soil Type B and Soil Type D.

In this part, 7 numbers of single earthquake named as DNFS1 until DNFS7 as well as 7 artificial ground motion to represent the repeated earthquake namely DNFR1 until DNFR7 has been subjected to the buildings constructed at Soil Type B and Soil Type D. It is to represent the moderate seismicity region especially at Sabah, Malaysia and the graph shows the result of IDR for the ground motion and its mean value.

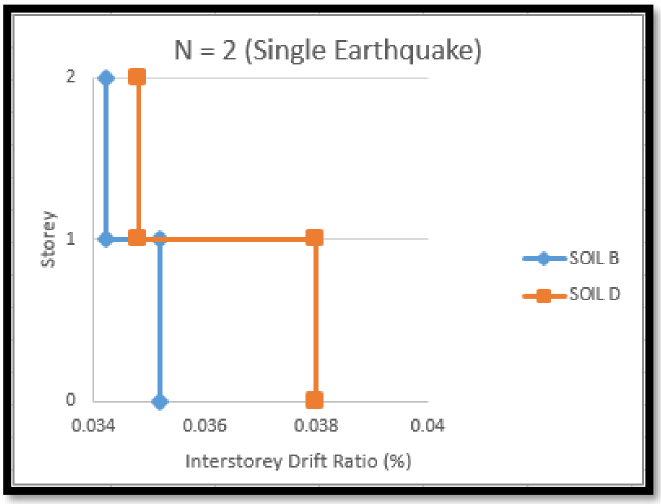
The seismic performance of each RC school model is evaluated based on the IDR (%). Generally, it is used to determine and evaluate the performance of the RC school buildings when subjected to single earthquake and repeated earthquake. IDR is the relative horizontal displacement between two adjacent storey normalized to its storey height. The equation used to determine the value of IDR stated as follow:

$$IDR = \frac{\Delta_i - \Delta_{i-1}}{h_i} \times 100 \tag{4.1}$$

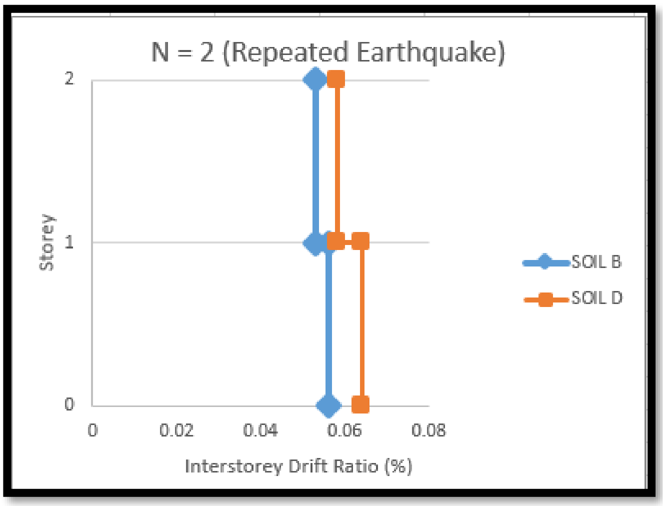
where Δ_i is the value of displacement for the each storey, and h_i is the storey height.

Basically, in Performance-Based Earthquake Engineering (PBEE), 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, denoted as IDR_{mean} (%) is considered from 7 numbers of single and repeated earthquake. This method is 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 repeated earthquakes is presented on Appendix H and Appendix J, respectively.

Figure 4.2 presents the distribution of IDR_{mean} along the height of the 2 storey RC school building when subjected to single and repeated earthquake in two different type of soil.



(a) Single earthquake



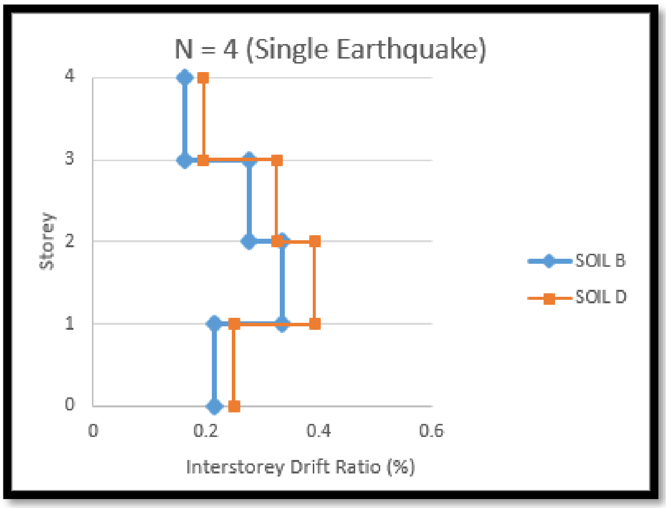
(a) Repeated earthquake

Figure 4.2 IDR_{mean} of 2 storey RC school building

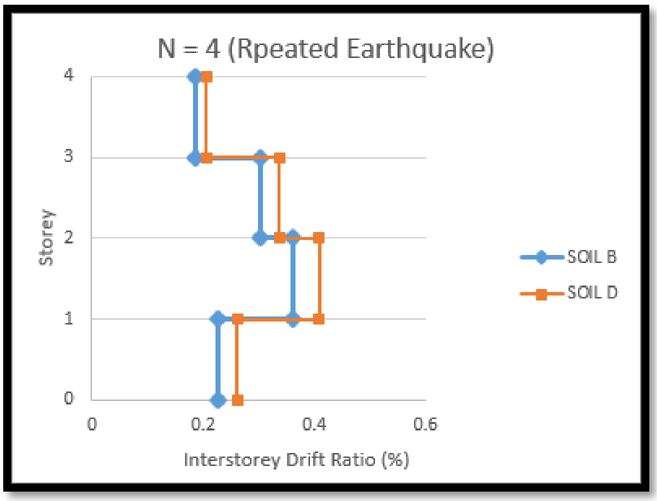
The lateral displacement was recorded and used to calculate the IDR value for each 7 different type of motions and the mean value is determined to get the final result as shown in the graph, known as IDR_{mean}. In Figure 4.2(a), the maximum IDR_{mean} on soil Type B is equal to 0.035% at the bottom level. For Soil Type D, the IDR_{mean} is equal to 0.038%, which is 8.5% higher than Soil Type B.

While for repeated earthquake, the IDR_{mean} for Soil Type B and Soil Type D is increasing from the single earthquake around 39% to 42%, respectively where the value for soil Type B is 0.056% and soil Type D is equal to 0.064%. The difference between both type of soil in single earthquake and repeated earthquake are 0.021% and 0.026%, respectively. This result is in good agreement with previous study that concluded that the IDR demand is higher when subjected to repeated earthquake compared to single earthquake, (Hatzigeorgiou and Liolios, 2010).

Figure 4.3 presents the distribution of IDR_{mean} along the height of the 4 storey RC school building when subjected to single and repeated earthquake in two different type of soil. The distribution of IDR_{mean} of 4 storey RC school building that have been subjected to the single earthquakes and repeated earthquakes is similar as the 2 storey frame. For single earthquake in Figure 4.3(a), IDR_{mean} for soil D is 14% greater than soil B with the value of 0.34% and 0.36%, respectively. Whereas the value is increasing for the repeated earthquake on both type of soil around 5% to 6% for Soil Type B and Soil Type D, respectively. In addition, the difference between two types of soil in single earthquake and repeated earthquake for both soil is 0.02%. It is strongly proven that the IDR_{mean} experienced by the frame that is subjected to repeated earthquake is higher compared to single earthquake and the value of IDR_{mean} in frame that designed on Soil Type D is always greater compared to Soil Type B.



(a) Single earthquake



(b) Repeated earthquake

Figure 4.3 IDRmean of 4 storey RC school building

CHAPTER 5

CONCLUSION

5.1 Introduction

This study presents the seismic performance of multi-storey and single-bay RC school building which consist of 2 storey and 4 storey in Sabah, Malaysia when subjected to single and repeated earthquake. The typical RC school building is designed according to BS8110 (1997) to represent the existing RC school building design and assumed to be built at two different type of soil, which is Soil Type B, and Soil Type D. A total of 7 single NFE ground motions and 7 artificial repeated ground motions had been used for nonlinear time history analysis (NTHA) by using SAP2000 program to obtain the mean value of interstorey drift ratio (IDR). There are few conclusions that can be drawn from his study as follows.

- In this study, the action of repeated earthquakes on frame tends to cause higher IDR compared to the action of single earthquakes. The magnitude of IDR is increasing from the single earthquakes to the repeated earthquakes around 5.0% to 6.0%. So, the behaviour of repeated earthquakes should be considered so that the building can survive the repeated earthquakes without high structural damages.

- The effect of soil type on seismic performance on RC school building are prove that the IDR for models on Soil Type D are always higher than Soil Type B. It is proven by the IDR value of models on Soil Type D which is around 14% higher than Soil Type B. The pattern are similar for both single and repeated earthquake. Therefore, soil type also should be taken in consideration for design as we can see soft soil will give high impact to the

RC school building and cause greater damages when subjected to repeated earthquakes.

5.2 Future Recommendations

The finding in this study is only applicable for the low rise building which is 2 and 4 storey. In addition, this study only took consideration in term of NFE. In order to improve this study, more research needs to be done. Some of the recommended studies for the future are as follow:

- Extend studies to high rise buildings, because more significant effect of earthquake is to the high rise buildings.
- The next study is to consider the FFE effect as we known that Malaysia especially, expose to the FFE earthquakes because it been surrounded by high seismicity regions.
- In order to make this study more realistic to Malaysia`s condition, the use of earthquake data for Malaysia is recommended.

In addition, research related to repeated earthquake can be carried out in future by considering different type of model such as hospital, residential or can be conducted for steel structure. As mention in this early study, school building always being a shelter for community when disaster occur in this country. So that, this building must be strong enough to support any load or force of the seismicity to make sure number of people died due to the earthquake can be reduced. The location to built the RC school building also should be emphasized because the soil type will give different level of impact when it is subjected to the earthquake.

Thus, repeated earthquake and type of soil should be taken in consideration of designing and evaluate of RC school building as it is proven that the effect of multiple tremors will give high impact to the RC school building in Malaysia and cause higher possibility of damages to the structure.

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APPENDIX A
NEARFIELD EARTHQUAKES (SINGLE EARTHQUAKES)

Table A.1: Scalling of real NFE to response spectrum for Soil Type B (N=2)

Fundamental period of vibration, T_1 = 0.17 sec			
No.	$Sa(T_1)$, g		Scale Factor (b/a)
	Original Record (a)	Response Spectrum (b)	
BNFS1	0.3527	0.4320	1.2248
BNFS2	0.3395	0.4320	1.2725
BNFS3	0.1792	0.4320	2.4107
BNFS4	0.6624	0.4320	0.6522
BNFS5	0.1146	0.4320	3.7696
BNFS6	0.5541	0.4320	0.7796
BNFS7	0.2982	0.4320	1.4487

Table A.2: Scalling of real NFE to response spectrum for Soil Type D (N=2)

Fundamental period of vibration, T_1 = 0.17 sec			
No.	$Sa(T_1)$, g		Scale Factor (b/a)
	Original Record (a)	Response Spectrum (b)	
DNFS1	0.8676	0.4860	0.5601
DNFS2	0.8190	0.4860	0.5934
DNFS3	0.2821	0.4860	1.7228
DNFS4	0.2415	0.4860	2.0124
DNFS5	0.1335	0.4860	3.6404
DNFS6	0.1432	0.4860	3.3939
DNFS7	0.1474	0.4860	3.2972

Table A.3: Scalling of real NFE to response spectrum for Soil Type B (N=4)

Fundamental period of vibration, T_1 = 0.53 sec			
No.	$Sa(T_1)$, g		Scale Factor (b/a)
	Original Record (a)	Response Spectrum (b)	
BNFS1	0.1762	0.4320	2.4518
BNFS2	0.5348	0.4320	0.8078
BNFS3	0.1581	0.4320	2.7324
BNFS4	0.6533	0.4320	0.6613
BNFS5	0.3034	0.4320	1.4239
BNFS6	0.4149	0.4320	1.0412
BNFS7	0.4320	0.4320	1.0000

Table A.4: Scalling of real NFE to response spectrum for Soil Type D (N=4)

Fundamental period of vibration, T_1 = 0.53 sec			
No.	$Sa(T_1)$, g		Scale Factor (b/a)
	Original Record (a)	Response Spectrum (b)	
DNFS1	0.6011	0.486	0.8085
DNFS2	0.4369	0.486	1.1124
DNFS3	0.4941	0.486	0.9836
DNFS4	0.4868	0.486	0.9984
DNFS5	0.1095	0.486	4.4384
DNFS6	0.1476	0.486	3.2927
DNFS7	0.1654	0.486	2.9383

(REPEATED EARTHQUAKES)

Table A.5: Combination of single earthquakes to generate repeated earthquakes for Soil Type B (N=2)

NO.	FORE-SHOCK	MAIN-SHOCK	AFTER-SHOCK
BNFR1	BNF1	BNF3	BNF5
BNFR2	BNF2	BNF4	BNF6
BNFR3	BNF1	BNF4	BNF5
BNFR4	BNF3	BNF2	BNF6
BNFR5	BNF5	BNF4	BNF3
BNFR6	BNF2	BNF6	BNF1
BNFR7	BNF4	BNF2	BNF6

Table A.6: Combination of single earthquakes to generate repeated earthquakes for Soil Type D (N=2)

NO.	FORE-SHOCK	MAIN-SHOCK	AFTER-SHOCK
DNFR1	DNF1	DNF3	DNF5
DNFR2	DNF2	DNF4	DNF6
DNFR3	DNF1	DNF4	DNF5
DNFR4	DNF3	DNF2	DNF6
DNFR5	DNF5	DNF4	DNF3
DNFR6	DNF2	DNF6	DNF1
DNFR7	DNF4	DNF2	DNF6

Table A.7: Combination of single earthquakes to generate repeated earthquakes for Soil Type B (N=4)

NO.	FORE-SHOCK	MAIN-SHOCK	AFTER-SHOCK
BNFR1	BNF1	BNF3	BNF5
BNFR2	BNF2	BNF4	BNF6
BNFR3	BNF1	BNF4	BNF5
BNFR4	BNF3	BNF2	BNF6
BNFR5	BNF5	BNF4	BNF3
BNFR6	BNF2	BNF6	BNF1
BNFR7	BNF4	BNF2	BNF6

Table A.8: Combination of single earthquakes to generate repeated earthquakes for Soil Type D (N=4)

NO.	FORE-SHOCK	MAIN-SHOCK	AFTER-SHOCK
DNFR1	DNF1	DNF3	DNF5
DNFR2	DNF2	DNF4	DNF6
DNFR3	DNF1	DNF4	DNF5
DNFR4	DNF3	DNF2	DNF6
DNFR5	DNF5	DNF4	DNF3
DNFR6	DNF2	DNF6	DNF1
DNFR7	DNF4	DNF2	DNF6

APPENDIX B
LATERAL DISPLACEMENT

(SINGLE EARTHQUAKE)

Table B.1: Lateral displacement of 2-storey RC school building (Soil Type B)

LEVEL	LATERAL DISPLACEMENT (m)							
	BNFS1	BNFS2	BNFS3	BNFS4	BNFS5	BNFS6	BNFS7	MEAN
2	0.00309	0.00384	0.0038	0.00303	0.00371	0.00329	0.00256	0.003331
1	0.00156	0.00195	0.0038	0.00156	0.00194	0.00164	0.0013	0.001689
0	0	0	0	0	0	0	0	0

Table B.2: Lateral displacement of 2-storey RC school building (Soil Type D)

LEVEL	LATERAL DISPLACEMENT (m)							
	DNFS1	DNFS2	DNFS3	DNFS4	DNFS5	DNFS6	DNFS7	MEAN
2	0.00405	0.00391	0.00437	0.00438	0.00114	0.000007	0.00035	0.002601
1	0.00205	0.00196	0.00236	0.00236	0.00057	0.000003	0.00018	0.001355
0	0	0	0	0	0	0	0	0

Table B.3: Lateral displacement of 4-storey RC school building (Soil Type B)

LEVEL	LATERAL DISPLACEMENT (m)							
	BNFS1	BNFS2	BNFS3	BNFS4	BNFS5	BNFS6	BNFS7	MEAN
4	0.03236	0.03762	0.03826	0.03487	0.03673	0.03595	0.03318	0.035567
3	0.02711	0.03239	0.03125	0.02913	0.0307	0.02984	0.02796	0.029769
2	0.01893	0.02094	0.02034	0.01937	0.02048	0.01961	0.01892	0.019799
1	0.00771	0.00817	0.00781	0.00749	0.00796	0.00743	0.00748	0.007721
0	0	0	0	0	0	0	0	0

Table B.4: Lateral displacement of 4-storey RC school building (Soil Type D)

LEVEL	LATERAL DISPLACEMENT (m)							
	DNFS1	DNFS2	DNFS3	DNFS4	DNFS5	DNFS6	DNFS7	MEAN
4	0.04136	0.04562	0.04341	0.03906	0.04376	0.03823	0.04162	0.041866
3	0.03463	0.03672	0.03648	0.03276	0.03621	0.03222	0.03468	0.034814
2	0.02324	0.02326	0.02461	0.02201	0.02377	0.02185	0.02306	0.023114
1	0.00908	0.00864	0.00975	0.00867	0.00915	0.00868	0.00899	0.008994
0	0	0	0	0	0	0	0	0

APPENDIX C
LATERAL DISPLACEMENT

(REPEATED EARTHQUAKE)

Table C.1: Lateral displacement of 2-storey RC school building (Soil Type B)

LEVEL	LATERAL DISPLACEMENT (m)							
	BNFR1	BNFR2	BNFR3	BNFR4	BNFR5	BNFR6	BNFR7	MEAN
2	0.00391	0.00393	0.00393	0.00391	0.00393	0.00397	0.00393	0.00393
1	0.00203	0.00202	0.00202	0.00203	0.00202	0.00201	0.00200	0.002019
0	0	0	0	0	0	0	0	0

Table C.2: Lateral displacement of 2-storey RC school building (Soil Type D)

LEVEL	LATERAL DISPLACEMENT (m)							
	DNFR1	DNFR2	DNFR3	DNFR4	DNFR5	DNFR6	DNFR7	MEAN
2	0.00437	0.00438	0.00438	0.00445	0.00438	0.00444	0.00445	0.004407
1	0.00236	0.00236	0.00236	0.00224	0.00236	0.00223	0.00224	0.002307
0	0	0	0	0	0	0	0	0

Table C.3: Lateral displacement of 4-storey RC school building (Soil Type B)

LEVEL	LATERAL DISPLACEMENT (m)							
	BNFR1	BNFR2	BNFR3	BNFR4	BNFR5	BNFR6	BNFR7	MEAN
4	0.03873	0.03908	0.03908	0.03764	0.03908	0.04088	0.03764	0.038876
3	0.0321	0.03235	0.03235	0.03141	0.03235	0.03334	0.03141	0.032187
2	0.02152	0.02135	0.02135	0.02094	0.02135	0.02141	0.02094	0.021266
1	0.00848	0.00824	0.00824	0.00817	0.00824	0.00796	0.00817	0.008214
0	0	0	0	0	0	0	0	0

Table C.4: Lateral displacement of 4-storey RC school building (Soil Type D)

LEVEL	LATERAL DISPLACEMENT (m)							
	DNFR1	DNFR2	DNFR3	DNFR4	DNFR5	DNFR6	DNFR7	MEAN
4	0.04373	0.04204	0.04204	0.04614	0.04204	0.04336	0.04614	0.043641
3	0.03682	0.03535	0.03535	0.03745	0.03535	0.03608	0.03745	0.036264
2	0.02491	0.02385	0.02385	0.02424	0.02385	0.02397	0.02424	0.02413
1	0.0099	0.00942	0.00942	0.00924	0.00942	0.00933	0.00924	0.009424
0	0	0	0	0	0	0	0	0

APPENDIX D
DISTRIBUTION OF LATERAL DISPLACEMENT

(SINGLE EARTHQUAKE)

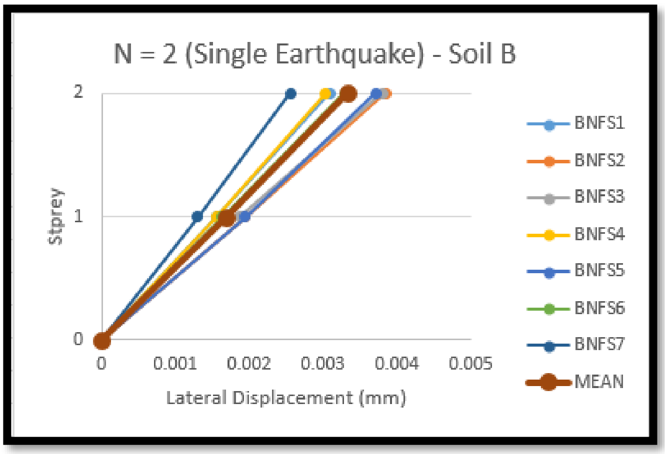

Figure D.1: Lateral displacement (m) of Soil Type B (N=2)

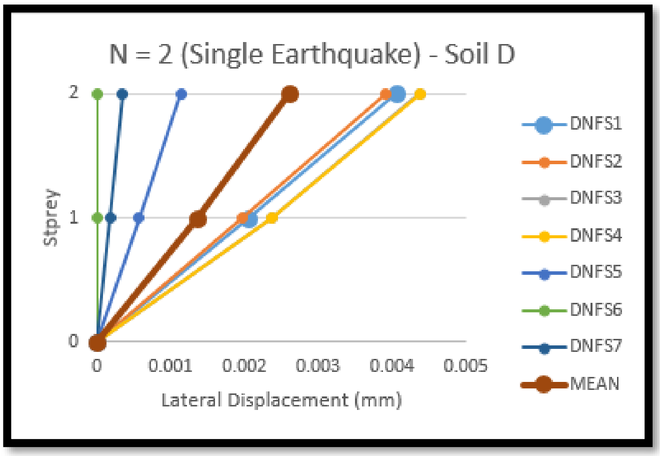

Figure D.2: Lateral displacement (m) of Soil Type D (N=2)

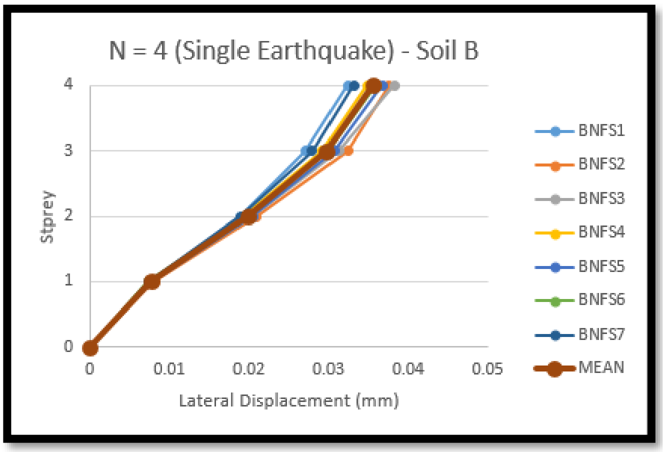

Figure D.3: Lateral displacement (m) of Soil Type B (N=4)

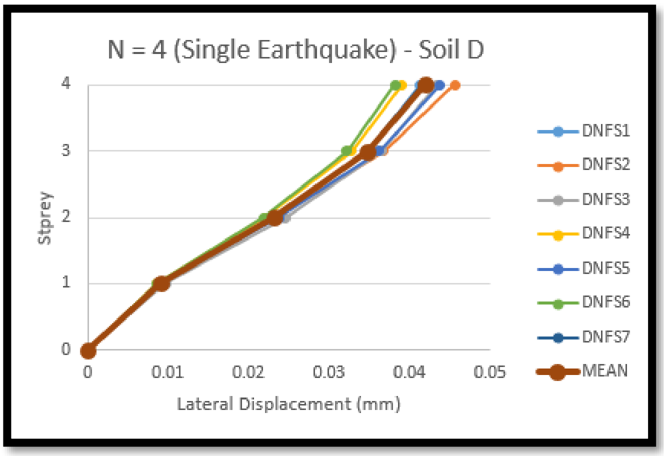

Figure D.4: Lateral displacement (m) of Soil Type D (N=4)

APPENDIX E
DISTRIBUTION OF LATERAL DISPLACEMENT

(REPEATED EARTHQUAKE)

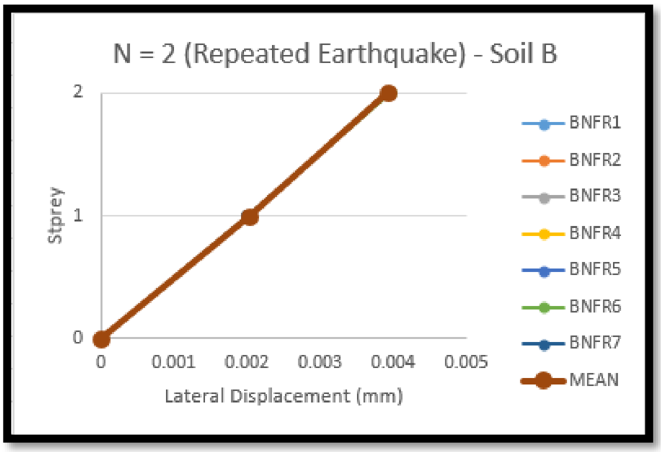

Figure E.1: Lateral Displacement (m) of Soil Type B (N=2)

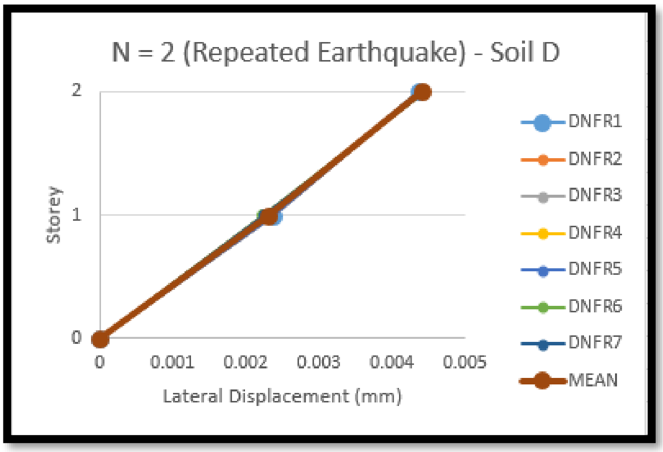

Figure E.2: Lateral Displacement (m) of Soil Type D (N=2)

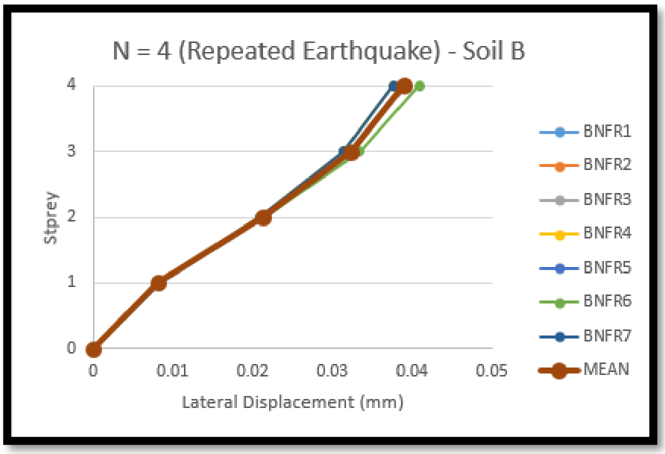

Figure E.3: Lateral Displacement (m) of Soil Type B (N=4)

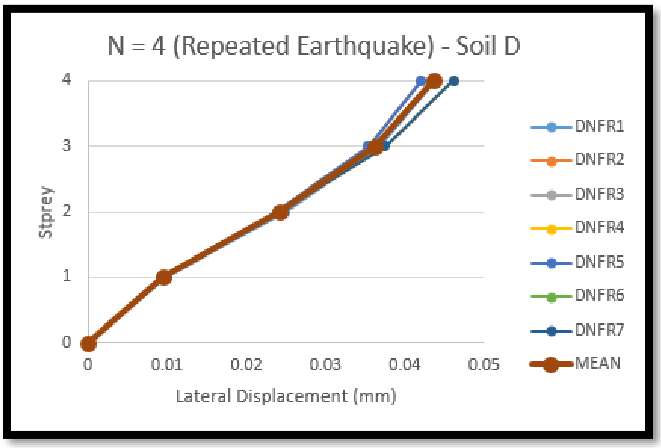

Figure E.4: Lateral Displacement (m) of Soil Type D (N=4)

APPENDIX F
INTERSTOREY DRIFT RATIO

(SINGLE EARTHQUAKE)

Table F.1: Interstorey drift ratio of 2-storey RC school building (Soil Type B)

LEVEL	LATERAL DISPLACEMENT (m)							
	BNFS1	BNFS2	BNFS3	BNFS4	BNFS5	BNFS6	BNFS7	MEAN
2	0.0425	0.0525	0.053611	0.040833	0.049167	0.045833	0.035	0.045635
1	0.043333	0.054167	0.051944	0.043333	0.053889	0.045556	0.036111	0.046905
0	0	0	0	0	0	0	0	0

Table F.2: Interstorey drift ratio of 2-storey RC school building (Soil Type D)

LEVEL	LATERAL DISPLACEMENT (m)							
	DNFS1	DNFS2	DNFS3	DNFS4	DNFS5	DNFS6	DNFS7	MEAN
2	0.055556	0.055556	0.055833	0.056111	0.015833	0.000111	0.004722	0.034817
1	0.056944	0.056944	0.065556	0.065556	0.015833	8.33E-05	0.005	0.037988
0	0	0	0	0	0	0	0	0

Table F.3: Interstorey drift ratio of 4-storey RC school building (Soil Type B)

LEVEL	LATERAL DISPLACEMENT (m)							
	BNFS1	BNFS2	BNFS3	BNFS4	BNFS5	BNFS6	BNFS7	MEAN
4	0.145833	0.145278	0.194722	0.159444	0.1675	0.169722	0.145	0.161071
3	0.227222	0.318056	0.303056	0.271111	0.283889	0.284167	0.251111	0.276944
2	0.311667	0.354722	0.348056	0.33	0.347778	0.338333	0.317778	0.335476
1	0.214167	0.226944	0.216944	0.208056	0.221111	0.206389	0.207778	0.214484
0	0	0	0	0	0	0	0	0

Table F.4: Interstorey drift ratio of 4-storey RC school building (Soil Type D)

LEVEL	LATERAL DISPLACEMENT (m)							
	DNFS1	DNFS2	DNFS3	DNFS4	DNFS5	DNFS6	DNFS7	MEAN
4	0.186944	0.247222	0.1925	0.175	0.209722	0.166944	0.192778	0.195873
3	0.316389	0.373889	0.329722	0.298611	0.345556	0.288056	0.322778	0.325
2	0.393333	0.406111	0.412778	0.370556	0.406111	0.365833	0.390833	0.392222
1	0.252222	0.24	0.270833	0.240833	0.254167	0.241111	0.249722	0.249841
0	0	0	0	0	0	0	0	0

APPENDIX G
INTERSTOREY DRIFT RATIO

(REPEATED EARTHQUAKE)

Table G.1: Interstorey drift ratio of 2-storey RC school building (Soil Type B)

LEVEL	LATERAL DISPLACEMENT (m)							
	BNFR1	BNFR2	BNFR3	BNFR4	BNFR5	BNFR6	BNFR7	MEAN
2	0.052222	0.053056	0.053056	0.052222	0.053056	0.054444	0.053611	0.053095
1	0.056389	0.056111	0.056111	0.056389	0.056111	0.055833	0.055556	0.056071
0	0	0	0	0	0	0	0	0

Table G.2: Interstorey drift ratio of 2-storey RC school building (Soil Type D)

LEVEL	LATERAL DISPLACEMENT (m)							
	DNFR1	DNFR2	DNFR3	DNFR4	DNFR5	DNFR6	DNFR7	MEAN
2	0.055833	0.055833	0.056111	0.061389	0.056111	0.061389	0.061389	0.058294
1	0.065556	0.065556	0.065556	0.062222	0.065556	0.061944	0.062222	0.064087
0	0	0	0	0	0	0	0	0

Table G.3: Interstorey drift ratio of 4-storey RC school building (Soil Type B)

LEVEL	LATERAL DISPLACEMENT (m)							
	BNFR1	BNFR2	BNFR3	BNFR4	BNFR5	BNFR6	BNFR7	MEAN
4	0.184167	0.186944	0.186944	0.173056	0.186944	0.209444	0.173056	0.185794
3	0.293889	0.305556	0.305556	0.290833	0.305556	0.331389	0.290833	0.303373
2	0.362222	0.364167	0.364167	0.354722	0.364167	0.373611	0.354722	0.36254
1	0.235556	0.228889	0.228889	0.226944	0.228889	0.221111	0.226944	0.228175
0	0	0	0	0	0	0	0	0

Table G.4: Interstorey drift ratio of 4-storey RC school building (Soil Type D)

LEVEL	LATERAL DISPLACEMENT (m)							
	DNFR1	DNFR2	DNFR3	DNFR4	DNFR5	DNFR6	DNFR7	MEAN
4	0.191944	0.185833	0.185833	0.241389	0.185833	0.202222	0.241389	0.204921
3	0.330833	0.319444	0.319444	0.366944	0.319444	0.336389	0.366944	0.337063
2	0.416944	0.400833	0.400833	0.416667	0.400833	0.406667	0.416667	0.408492
1	0.275	0.261667	0.261667	0.256667	0.261667	0.259167	0.256667	0.261786
0	0	0	0	0	0	0	0	0

APPENDIX H
DISTRIBUTION OF INTERSTOREY DRIFT RATIO

(SINGLE EARTHQUAKE)

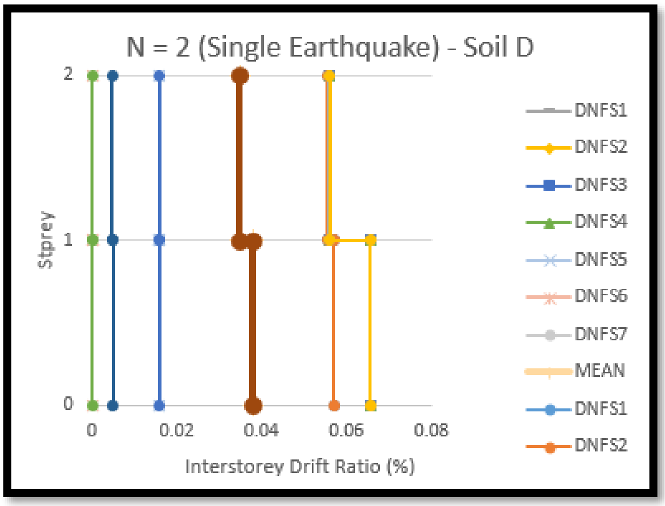

Figure H.1: Interstorey Drift Ratio (IDR) (%) of Soil Type B (N=2)

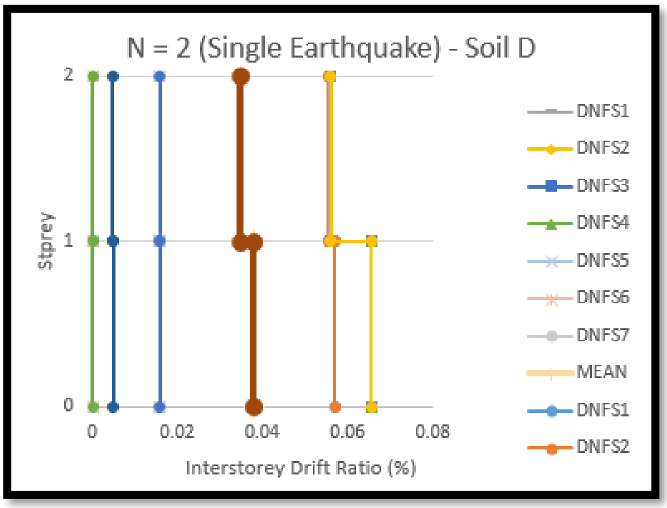

Figure H.2: Interstorey Drift Ratio (IDR) (%) of Soil Type D (N=2)

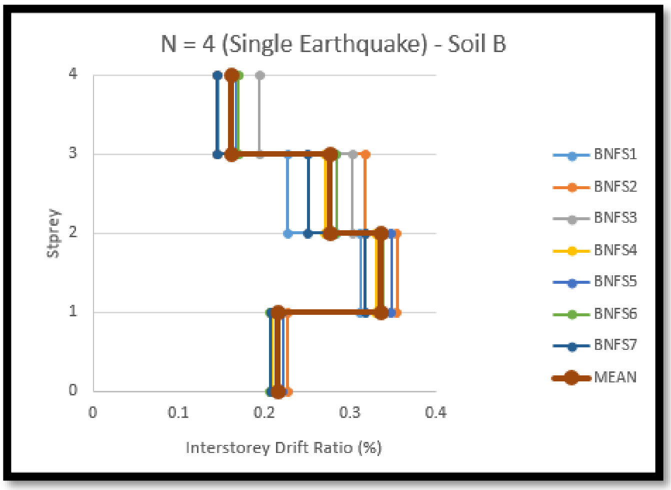

Figure H.3: Interstorey Drift Ratio (IDR) (%) of Soil Type B (N=4)

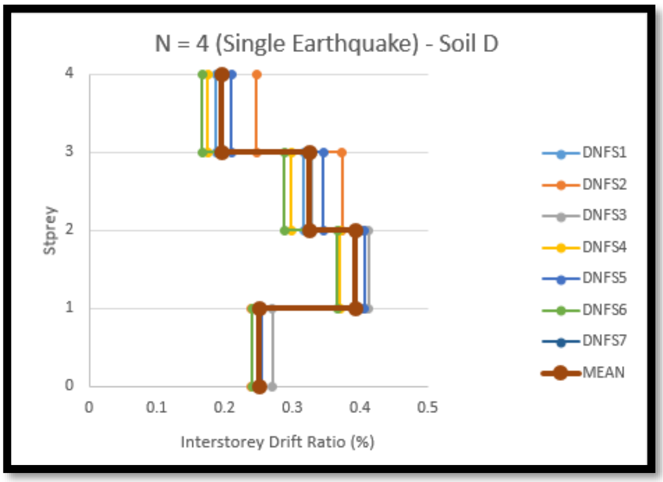

Figure H.4: Interstorey Drift Ratio (IDR) (%) of Soil Type D (N=4)

APPENDIX J
DISTRIBUTION OF INTERSTOREY DRIFT RATIO

(REPEATED EARTHQUAKE)

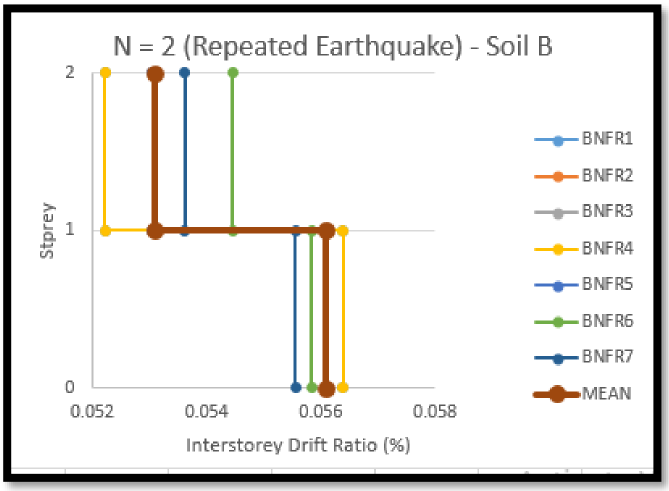

Figure J.1: Interstorey Drift Ratio (IDR) (%) of Soil Type B (N=2)

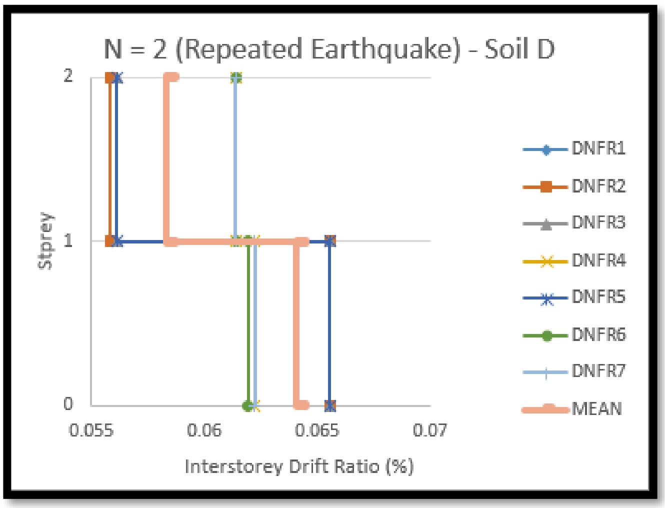

Figure J.2: Interstorey Drift Ratio (IDR) (%) of Soil Type D (N=2)

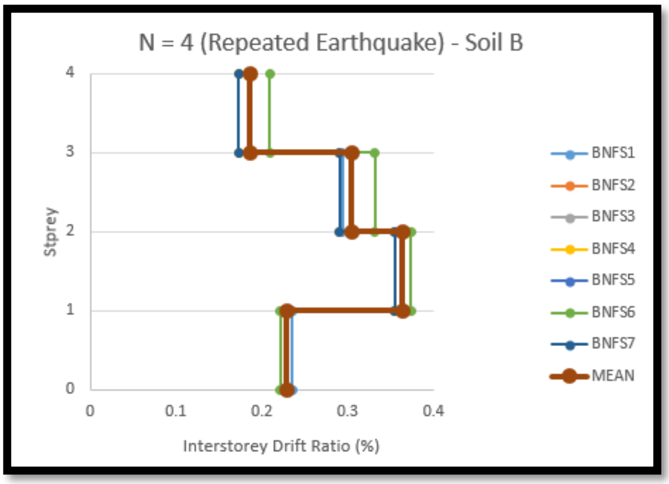

Figure J.3: Interstorey Drift Ratio (IDR) (%) of Soil Type B (N=4)

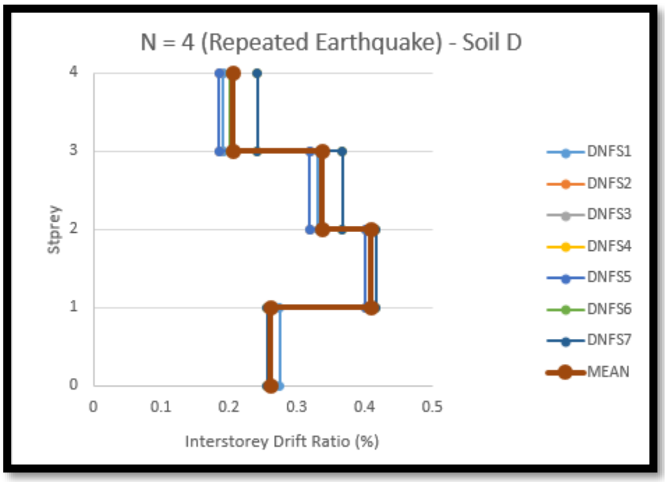

Figure J.4: Interstorey Drift Ratio (IDR) (%) of Soil Type D (N=4)