SEISMIC DESIGN FOR REINFORCED CONCRETE HOSPITAL BUILDING INFLUENCED BY LEVEL OF PEAK GROUND ACCELERATION AND CLASS OF DUCTILITY

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SEISMIC DESIGN FOR REINFORCED CONCRETE HOSPITAL BUILDING INFLUENCED BY LEVEL OF PEAK GROUND ACCELERATION AND CLASS OF DUCTILITY

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ABSTRAK

Kekerapan berlakunya gempa bumi seperti gempa di Sumatera-Andaman pada 26 Disember 2004, gempa bumi di Nias pada 28 Mac 2005, dan gempa bumi di Bengkulu pada 12 Disember 2007 telah mempengaruhi beberapa kejadian gempa di Semenanjung Malaysia. Antara tempat yang terkesan ialah di Bukit Tinggi pada 30 November 2007 hingga 25 Mei 2008, Jerantut pada 17 Mac 2009, Manjong pada 29 April 2009, dan Kuala Pilah pada 29 dan 30 November 2009. Manakala di sebelah Timur Malaysia terutamanya di Sabah, ia telah diketahuai umum sebagai kawasan yang terdedah kepada aktiviti gempa bumi. Oleh itu, ianya terbuktilah bahawa Malaysia tidak sepenuhnya bebas daripada berlakunya bencan gempa bum ini sama ada di semenanjung Malaysia mahupun di timur Malaysia. Pada tahun 2009, Jabatan Kerja Awam Malaysia (JKR) telah berpendapat untuk mempertimbangkan input rekabentuk seismik di bangunan-bangunan baru yang terletak di zon gempa yang berisiko sederhana dan tinggi. Kepentingan untuk menilai kos perlaksaan rekabentuk seismik ini amatlah dititikberatkan. Oleh itu, kajian ini telah membincangkan tentang reka bentuk dan analisa seismik terhadap bangunan hospital yang mengambil kira pelbagai nilai pergerakan tanah dan kelas kemuluran bangunan yang berlainan. Di akhir kajian ini, jumlah besi yang diperlukan untuk membina bangunan seismik ini telah dibandingkan dengan jumlah besi yang diperlukan untuk reka bentuk tanpa mempertimbangkan parameter seismik. Enam model bangunan hospital dengan pertimbangan kelas PGA dan kemuluran yang berbeza telah diambil kira, iaitu bangunan bukan seismik, kemuluran sederhana dengan PGA 0.04g, 0.08g, 0.12g, 0.16g dan kemuluran rendah dengan PGA 0.04g. Untuk magnitud yang mempunyai PGA yang berbeza, hasil menunjukkan bahawa perbezaan peratusan besi yang diperlukan berbanding dengan reka bentuk bukan seismik untuk rasuk dan lajur seluruh bangunan telah meningkat dari 6%, 116%, 257%, dan 290% untuk PGA sama dengan 0.04g, 0.08g, 0.12g dan 0.16g. Manakala bagi kelas kemuluran yang berlainan, keputusan menunjukkan bahawa perbezaan peratusan pengukuhan besi yang diperlukan berbanding dengan reka bentuk bukan seismik telah meningkat dari 6% hingga 145% untuk kelas kemuluran sederhana dan kemuluran rendah masing-masing. Natijahnya, nilai PGA dan kelas kemuluran sesebuah bangunan telah memberi kesan yang penting kepada jumlah keseluruhan besi yang diperlukan. Oleh itu, dua parameter tersebut haruslah dipertimbangkan dalam merekabentuk sesebuah bangunan seismik di Malaysia.

ABSTRACT

A series of earthquakes such as Sumatra-Andaman earthquake on 26 December 2004, Nias earthquake on 28 March 2005, and Bengkulu earthquake on 12 December 2007 had influences to a series of subsequence local earthquake in Peninsular Malaysia. Some of the local earthquake that had been affected are at Bukit Tinggi on 30 November 2007 to 25 May 2008, Jerantut on 17 March 2009, Manjong on 29 April 2009, and Kuala Pilah on 29 and 30 November 2009. While in East of Malaysia especially Sabah, it is locally known as earthquake prone region. Hence it can be concluded that Malaysia is not totally free from seismic activities either in peninsular Malaysia or at the east of Malaysia. In 2009, Malaysia Public Work of Department (PWD) felt it was worthwhile to consider seismic design input in new building which are located in medium to high risk earthquake zone. The effect of seismic design implementation on cost of materials is became an important topic to be investigated. In relation to that, this study discusses on the seismic design of reinforce concrete hospital building with consideration of different magnitude of Peak Ground Acceleration (PGA) and different class of ductility. The outcome of the design is the comparison on the amount of steel reinforcement required that is obtained from two different parameters mentioned above compared to non-seismic design. Six models of hospital buildings with consideration of different PGA and ductility class are considered, namely, non-seismic building, medium ductility with PGA of 0.04g, 0.08g, 0.12g, 0.16g and low ductility with PGA of 0.04g. For different magnitude of PGA, the results shows that the percentage difference of steel reinforcement required compared to non-seismic design for beam and column of the whole building had increased from 6%, 116%, 257%, and 290% for PGA equals to 0.04g, 0.08g, 0.12g, and 0.16g respectively. While for different class of ductility, the results shows that the percentage difference of steel reinforcement required compared to non-seismic design had increased from 6% to 145% for ductility class medium and ductility class low respectively. Thus, magnitude of PGA and class of ductility of structure give significant effect to overall amount of steel reinforcement required. Hence, it should be considered in designing a seismic building.

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LIST OF SYMBOLS

a_g	Design ground acceleration
a_{gR}	Reference peak ground acceleration
As_{prov}	Total area of steel provided
As_{req}	Total area of steel required
d_{bL}	Diameter of longitudinal bar
d_{bw}	Diameter of shear or confinement bar
F_b	Base shear force
f_{cd}	Design value of concrete compressive strength
f_{ck}	Characteristic cylinder strength of concrete
F_i	Lateral load on storey
f_y	Yield strength of reinforcement
g	Acceleration due to gravity, m/s^2
G_k	Dead load
H	Storey height
M	Bending moment
m	mass of structure
M_{Rb}	Design moment resistance of beam
M_{Rc}	Design moment resistance of column
M_w	Magnitude of earthquake intensity
n	Number of storey
q	Behaviour factor
Q_k	Live load
S	Soil factor
$Sd(T_1)$	Ordinate of the design spectrum at period
T_1	Fundamental period of vibration
T_B	Lower limit of the period of the constant spectral acceleration
T_C	Lower limit of the period of the constant spectral acceleration
T_D	Beginning of the constant displacement response range of the spectrum
V	Beginning of the constant displacement response range of the spectrum

LIST OF ABBREVIATIONS

DCH	Ductility class high
DCL	Ductility class low
DCM	Ductility class medium
$K_{elastic}$	Elastic stiffness
PGA	Peak ground acceleration

CHAPTER 1

INTRODUCTION

1.1 Background

Earthquake is considered as one of the most devastated natural disasters which had cause many human fatalities and economic losses. Earthquake happens when two blocks of the earth suddenly slip past one another, where the slip is called the fault plane. The location below the earth's surface where the earthquake starts is called the hypocentre, and the location directly above it on the surface of the earth is called epicentre. According to (Natoli, 2005) earthquake intensity generally decreases with increasing distance away from epicentre because seismic wave amplitude gradually die down as the waves travel through the earth.

An earthquake results from the sudden release of energy stored in the lithosphere by the continuous motion of plates (Achache, 1986). Layers of the earth which are the crust and mantle made up a thin layer of tectonic plates at the surface of the earth. The boundaries of the tectonic plates are made up of many faults, since the edges of the plates are rough they get stuck while the rest of the plates keep moving. When the force of the moving plates overcomes the friction of the edges of the faults, the stored up energy will be released which will reach the earth surface and there is an earthquake.

According to Jabatan Mineral dan Geosains Malaysia (JMGM), Malaysia is considered as a country that has relatively low seismicity except for the state of Sabah where earthquake is locally known to occur (MOSTI, 2009). Because of that, Malaysia had not consider seismic load in structural design. This is due to our geographical location which are situated on the stable part of Sundaland and located far from active seismic fault region.

However, the low seismic hazard in Malaysia cannot be taken lightly as Malaysia is surrounded by high seismicity regions from neighbouring countries such as Indonesia and Philippine. According to Pappin et al. (2011), these high seismicity regions is strongly associated with the subduction zones between the Eurasian and Philippines plates at the east part as shown in Figure 1.1.

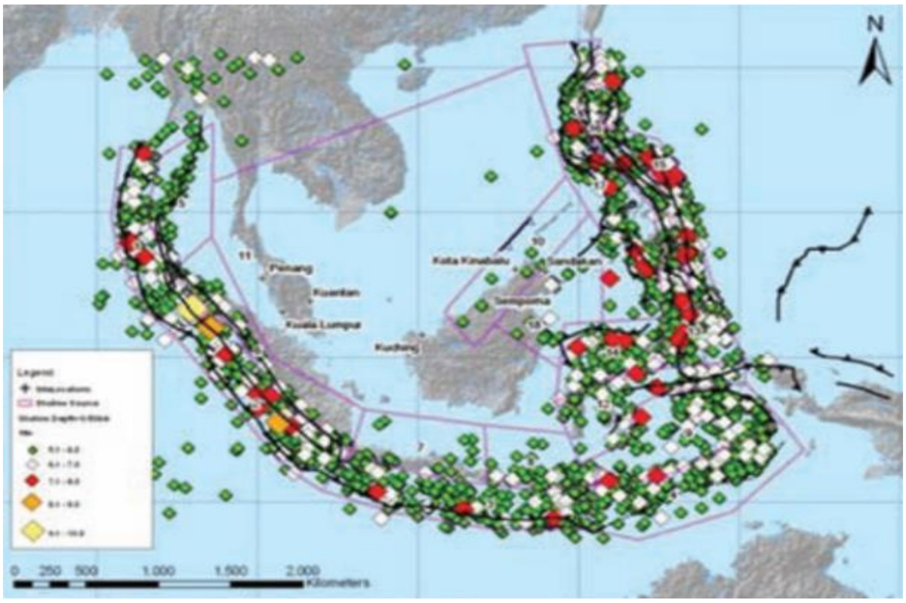

Figure 1.1 Earthquake events since 1972 to a depth of 50 km (Pappin et al., 2011)

Therefore, Malaysia will have a certain risk of earthquake coming from the regions especially in the west coast of peninsular Malaysia and Sabah. This is proven when Malaysia had been affected by a severe earthquake with magnitude M_w 9.0 that had struck Aceh, Indonesia on 26 December 2004. The ground tremor from this very strong earthquake could be felt within Peninsular Malaysia, where local earthquake had been reported in Bukit Tinggi with magnitude M_w up to 3.5 (MOSTI, 2009).

While at the east of Malaysia, particularly at state of Sabah is more prone to seismic activity. One of the worst earthquake occurred in Lahad Datu with magnitude of M_w 5.8 on 1976 which had caused damages to school building and even worse is the Ranau earthquake with magnitude of M_w 5.9 on June 2015 that had cause injuries and death to the people. This is supported by Harith (2016), where the statistics for an updated earthquake recorded from 1884 through 2016 represented by magnitude indicates a large increment of earthquake events for the last 140 years as shown in Figure 1.2.

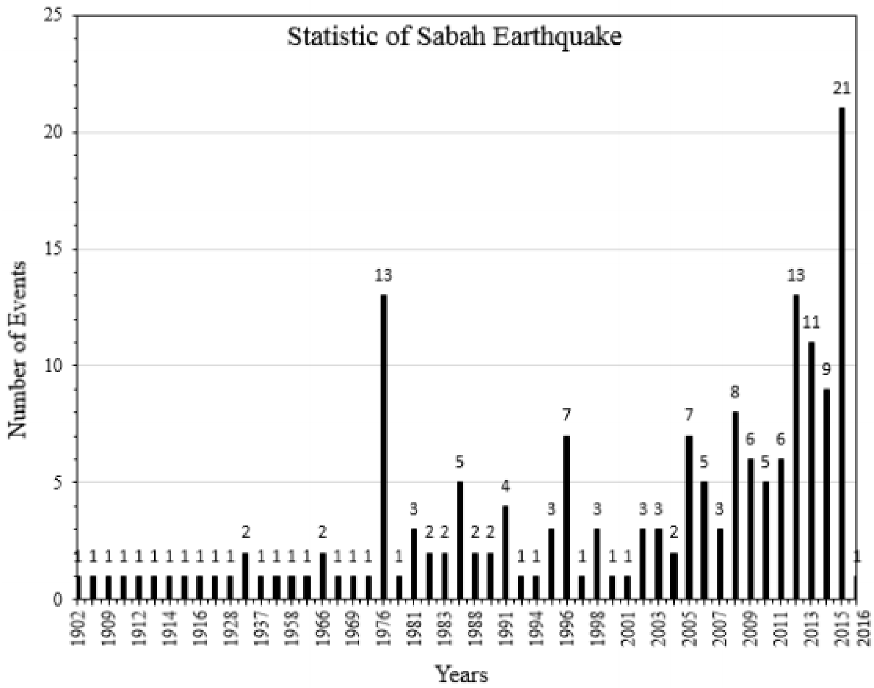

Figure 1.2 Number of local earthquakes with a magnitude greater than 2.0 reported in each decade (1900-2016) around Sabah, (Harith, 2016)

Having affected by the earthquakes, both Peninsular and Eastern part of Malaysia had been aware of the seismic hazard and necessities of applying seismic design on new buildings. Although Peninsular Malaysia has a very low seismic risk, the damage potential could not be neglected as a large earthquake from neighbouring countries could create considerably ground motion over western part of Peninsular Malaysia. Some of the degree of risk faced is when the geographical condition is composed of limestone it may cause to sinkhole development. The topographical condition with steep slopes may cause movements or landslide which may lead to damages and fatalities. On top of that, it is important to be noted that the physical size of an earthquake is not the only factor that cause damages, but it also depends on other factors such as where and when an earthquake occurred and the population density in the area concerned. The necessities on seismic research based on previous seismic activity in Malaysia is supported by Adiyanto (2016), where the M_w 5.8 Ranau earthquake in sabah has become the strongest reason on why the researches related to earthquake in Malaysia is always relevant.

There are a few factors which influencing the seismic design such as the site location, soil type, peak ground acceleration (PGA), materials, type of structures, ductility, stiffness and behaviour factor, *q* (Adiyanto, 2016). This study focused on the effect of different magnitude of PGA and the types of ductility class to seismic design of reinforced concrete (RC) hospital building. These are two important parameters in specifying earthquake actions for seismic design which will determine the total amount of steel required in seismic design. In overall, this study will be significant in reducing damages of element in a structure caused by earthquake as well as determining the influences of different level of PGA and ductility class to seismic design.

1.2 Problem Statement

A study on seismotectonic setting by JMGM had developed an Earthquake Hazard Maps for Malaysia as shown in Figure 1.2. A total of 5 risk zones had been identified based on Maximum Mercalli Intensity (MMI) and the PGA data. In general, Peninsular Malaysia appeared to be a region that has inactive plate and is generally stable from seismic activity. However, a series of large earthquake at the surrounding of neighbouring active plate had changed the tectonic setting in the Southeast Asia region including Peninsular Malaysia.

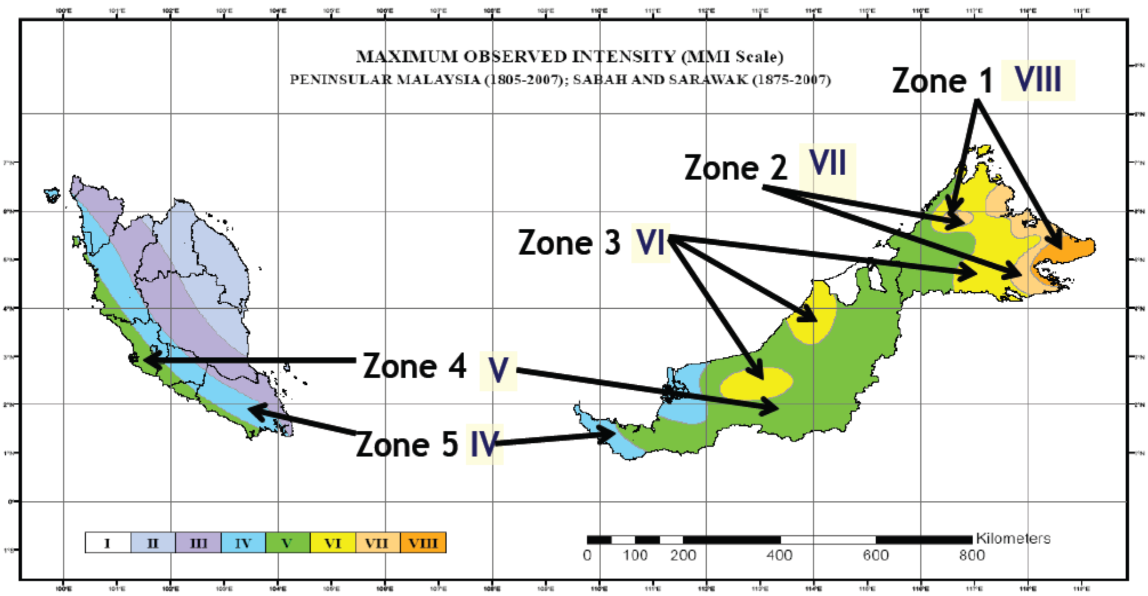

Figure 1.2 Earthquake Hazard Zonation (MOSTI, 2009)

According to MOSTI (2011), it is believed that the reoccurrence of local earthquakes happen in peninsular Malaysia are related to stress effected from the southern Sumatra earthquake. A series of these earthquakes are Sumatra-Andaman earthquake on 26 December 2004, Nias earthquake on 28 March 2005, and Bengkulu earthquake on 12 December 2007. Some of the local earthquake that had been affected are at Bukit Tinggi on 30 November 2007 to 25 May 2008, Jerantut on 17 March 2009, Manjong on 29 April 2009, and Kuala Pilah on 29 and 30 November 2009 (MOSTI, 2009). According to Pappin et al. (2011) for location on the western of peninsular Malaysia, the seismic hazard has become higher due to the significant seismic activity under Sumatra which lead to the important conclusions that building above 10 storey especially those founded in deep or soft soil on the western side of Peninsular Malaysia should consider seismic loading as part of their design.

While in East of Malaysia especially Sabah, it is locally known as earthquake prone region. In accordance to Tjia (2007), Sabah experienced moderate seismicity in the active Mensaban, and Lobou-Lobou fault zones located in Kundasang and Ranau which have brought earthquakes that caused light damage to infrastructures. Figure 1.3 illustrated the major fault around Sabah. Hence it can be concluded that Malaysia is not totally free from seismic activities.

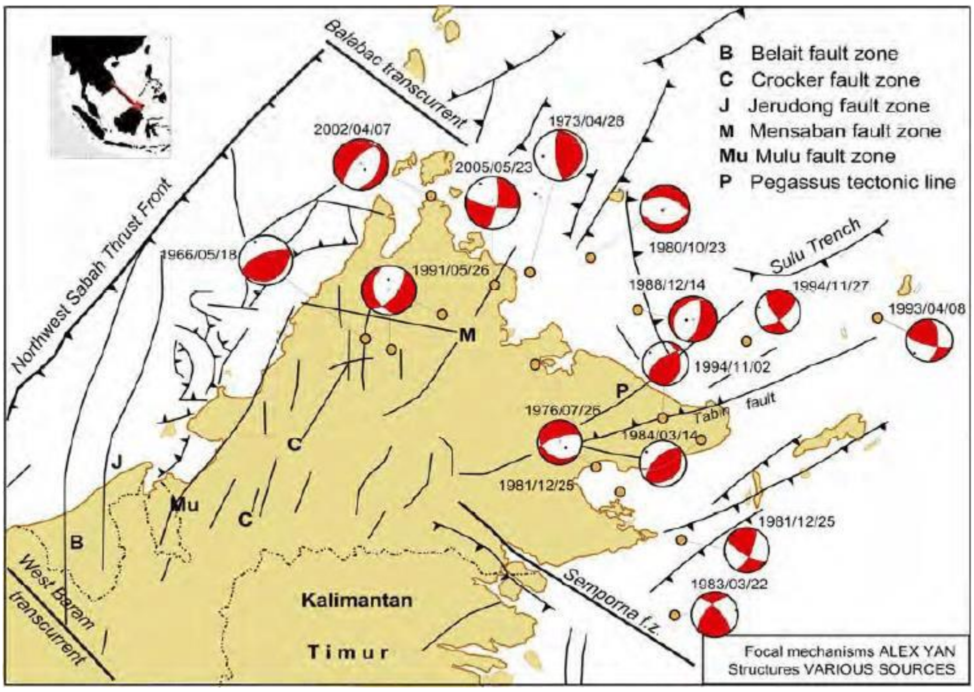

Figure1.3 Seismic geometry of local earthquake around Sabah (Alexander et al., 2006)

The life threatening and damages resulted from the seismic activity makes people concern on how safe their buildings are when subjected to any unexpected seismic movement. This is due to the fact that current practice in structural design does not consider any seismic provision in buildings. This is proven when there is less than one percent of buildings in Malaysia are seismic resistant (Majid, 2009). Therefore, in 2009 Malaysia Public Work of Department felt it was worthwhile to consider seismic design input in new building which are located in medium to high risk earthquake zone (MOSTI, 2009). The effect of seismic design implementation on cost of materials is became an intensity topic to be investigated.

In relation to that, this study discusses on the seismic design of RC hospital building with consideration of different magnitude of PGA and different class of ductility. The outcome of the design is the comparison on the amount of steel reinforcement required that is obtained from two different parameters mentioned above.

1.3 Main Objectives

The objectives of this study are:

i. To study the influences of magnitude of PGA on the amount of steel reinforcement

ii. To study the influences of class of ductility on the amount of steel reinforcement

1.4 Scope and Limitation Research

This study covered and focused in the following aspect:

i. A 6 storey RC hospital building served as the main model.

ii. Four different magnitude of PGA equal to 0.04g, 0.08g, 0.12g, and 0.16g had been considered for design.

iii. Models had been designed based on Eurocodes 8 (2004) for ductility class low and ductility class medium.

iv. Tekla Structural Design software had been used for analysis and design.

v. All models were considered to be built on Soil Type D. The design had been conducted for compressive strength of concrete, f_{cu} equal to 30 N/mm^2.

vi. The result are discussed in term of comparison of steel required as reinforcement influenced by magnitude of PGA and ductility class.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Developing a seismic design for structures requires a number of important factors that should be analyse well. This study adopted Eurocode 8 as a guideline in overall design analysis. Eurocode 8 is a design code of practice for earthquake resistance design structures. These code emphasise on the basis of seismic design to protecting human life, minimizing structures damages and ensuring operational continuity of constructions during and after earthquake. In order to get a clear understanding about this study, this chapter will discusses regarding the characteristics of seismic design and other topics which relevant to it such as Ground motion, Ductility Factor, Strong Beam-Weak Column Concept, and seismograph network in Malaysia.

2.2 Ground motion

The crux of specifying earthquake actions for seismic design lies in estimating the ground motions caused by earthquakes (Elghazouli, 2009). Ground motion is the movement of the earth's surface from earthquakes or explosions. Ground motions is produced by waves that are generated by sudden slip on a fault and travel through the earth and along its surface. As aware, the shaking cause by the seismic waves can cause damage to building or cause the building to collapse.

When an earthquake occur, its effect can depend on types of faulting that generate the seismic wave. Refer to Figure 2.1, there are generally 3 types of faults depending on the direction of a relative displacement or slip in the fault which are strike slip fault, normal fault and reverse fault. Dip slip fault which comprises of normal faults and reverse fault happens when one blocks moves vertically respect to the other while strike slip fault happens when adjacent block moves horizontally past one another. Reverse faulting earthquakes tending to generate larger ground motions than either strike-slip or normal faulting events (Bommer et al., 2003).

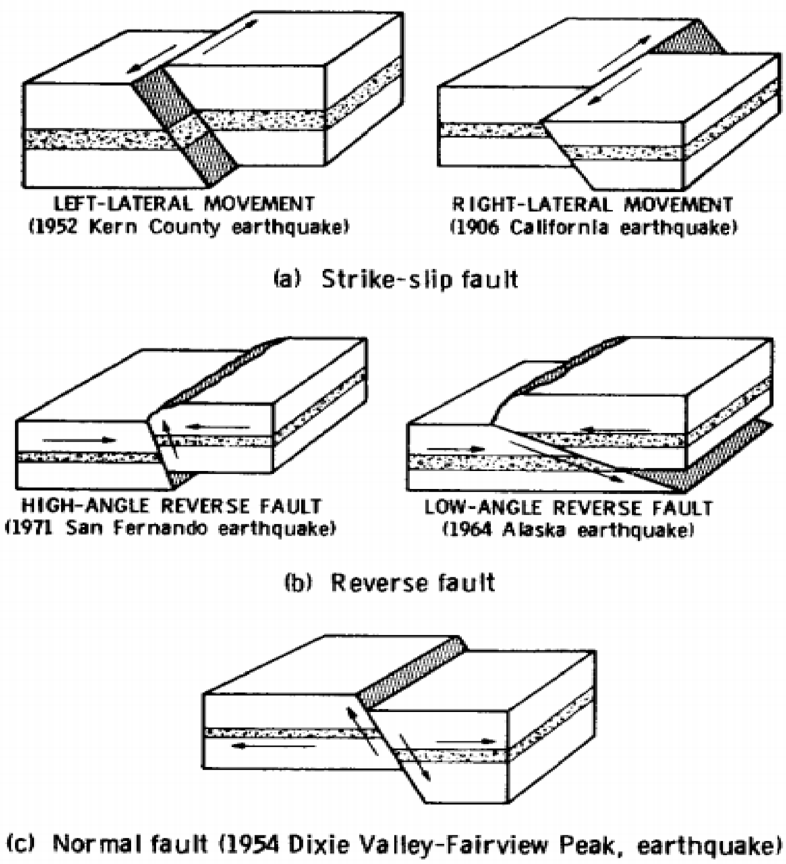

Figure 2.1 Faults types

According to several researchers, ground motion parameters can be expressed in several categorizations. These are presented in Table 1.1 while Figure 2.2 shows example of the ground motion parameters presented in graph. The peak ground acceleration is the most widely used measure of the strength of ground shaking (S.D. Werner, 1976). Peak Ground Acceleration (PGA) is the maximum force applied to a high frequency system and the data regarding strength levels of past earthquake motions is provided in terms of peak accelerations (Elghazouli, 2009).

Table 2.1 Category and ground motion parameter

Category	Parameters
1. Peak values	Peak acceleration, velocity, displacement, response spectrum
2. Earthquake durations	Total duration, effective durations
3. Frequency contents	Predominant spectral period, number of zero crossing
4. Intensity values	Arias, Housner, earth power, root mean square, CAV
5.Combinations	Destructiveness potential, mean input energy, damage capacity

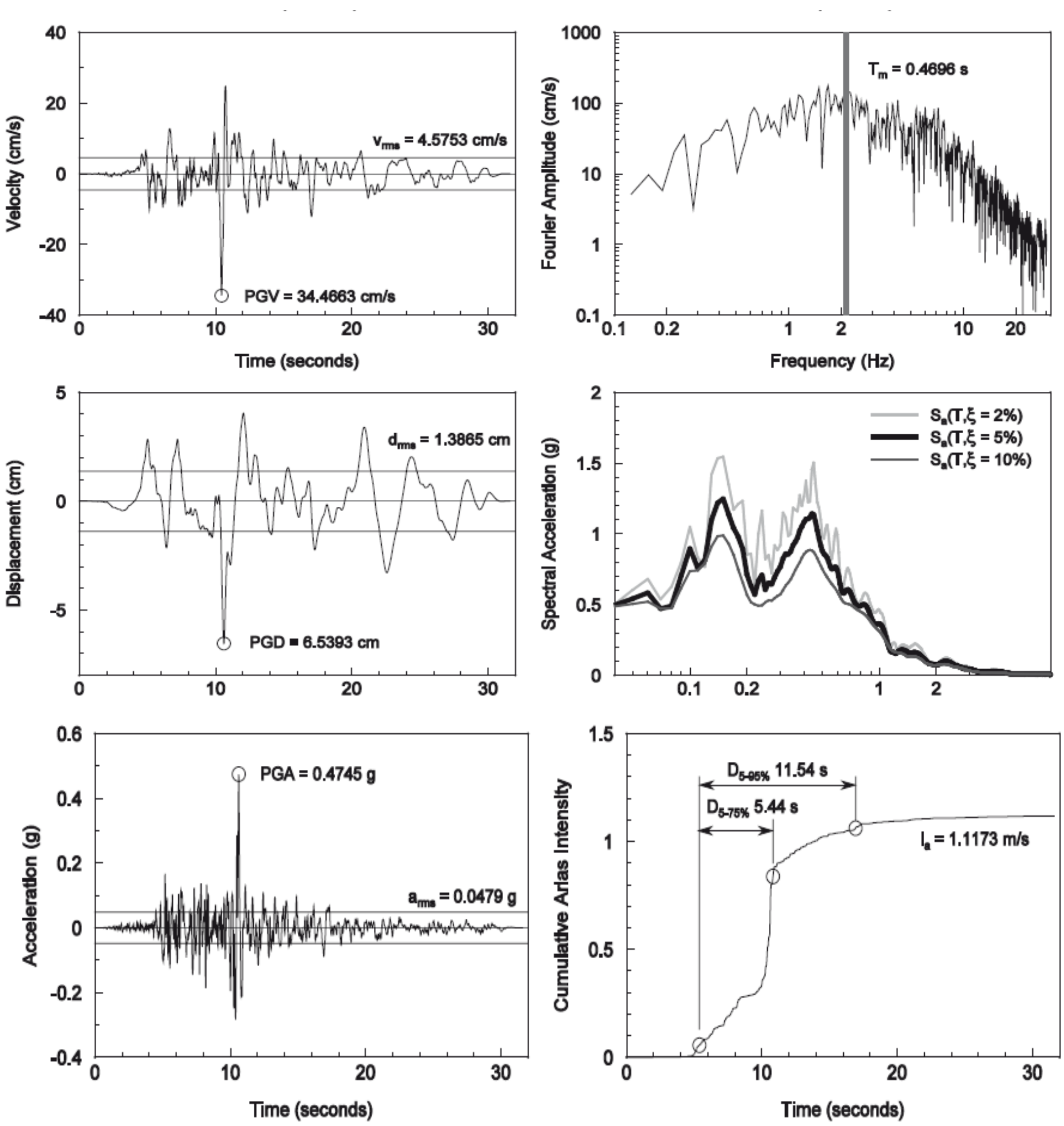

Figure 2.2 Examples of ground motion parameters

In designing a seismic resistant building, three most important characteristics of ground shaking are the value of PGA, the duration of strong shaking and the frequency content of the shaking. Recorded peak ground accelerations of damaging earthquakes range from 0.2 g to over 1.0 g where g is the acceleration due to gravity (Andrew, 2008). As Malaysia is considered as having a low seismic activities, there is limited historical data of appropriate ground motion for seismic design. Many researchers has been devoted to investigating the characteristics of ground motion in Malaysia. The result from a research project to develop the macrozonation map for Malaysia was conducted by Universiti Teknologi Malaysia, Universiti Sains Malaysia and Universiti Teknologi Mara had produce the value of PGA for all regions in Malaysia. The PGA map for Peninsular and Eastern Malaysia is shown in Figure 2.3 and Figure 2.4.

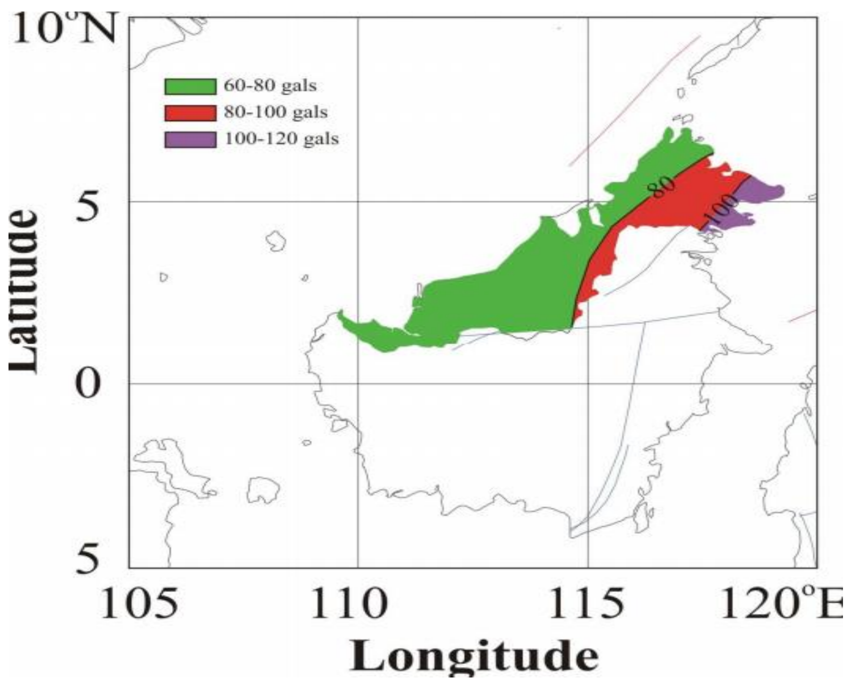

Figure 2.3 PGA map of East Malaysia with 500 years return period

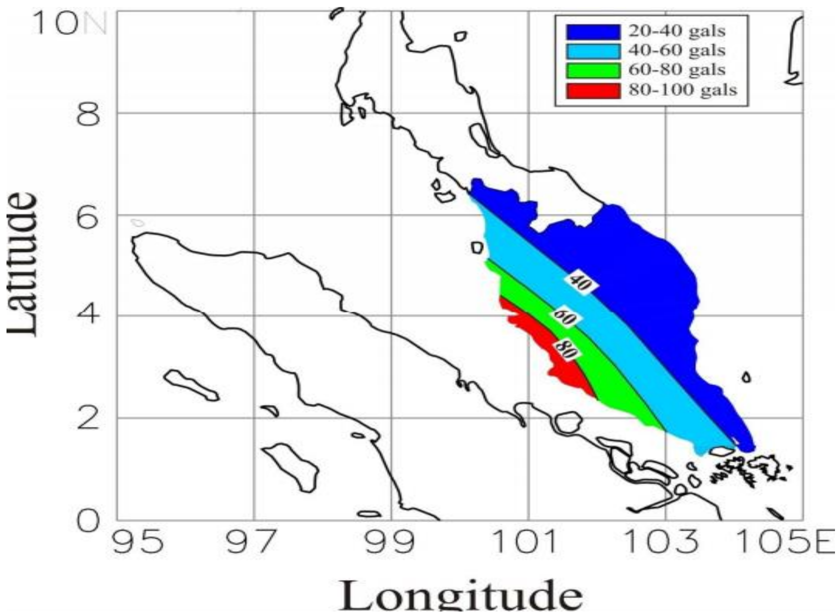

Figure 2.4 PGA map of Peninsular Malaysia with 500 years return period

Ramli et al. (2017) studied on the costing impact to the adoption of seismic action in the designs. The comparison is made in term of total reinforcement requirements between design practice using Eurocode 2 (2004) and similar design including requirements in Eurocode 8 (2004) with different class and PGA. The analysis and design is based on structure frame element (beams and columns) for five and ten storey buildings. For non-seismic design, it is designed based on Eurocode 2 (2004) and consider only live load and dead load. For medium ductility class, PGA values of 0.08g and 0.14g are chosen while for Ductility Class Low (DCL), PGA value chosen is 0.06g. Based on this study, it has been concluded that, the higher value of PGA will increase the total cost for the whole project. In this study, the states of Kedah and Johor with PGA of 0.06g showed the lowest cost, while Penang and Johore with PGA of 0.08g has higher cost. Ramli et al. (2017) also added that with consideration of seismic design, the total cost of a project will increase, but the cost of repair and maintenance maybe reduced.

The influences of different value of PGA in design analysis also had been investigated by Adiyanto and A. Majid (2014). This study investigated on the difference of steel reinforcement and concrete required when seismic provision is considered in a two storey general office building with consideration of various level of PGA. Since this study only consider DCL, the behaviour factor is equal to 1.5 as in Eurocode 8. The value of PGA used are 0.02 g, 0.06g, and 0.12g. All frames had been evaluated using nonlinear time history analysis. Some conclusion had been made by the author the total volume of concrete is strongly influenced by the level of reference peak ground acceleration, a_{gR} especially for column element. This is due to rapid increment of bending moment to be resisted by column when seismic load is considered in design.

From the previous research, it is clear that the effect of different value of PGA gives significant effect to the final outcome of the analysis. Therefore, these ground motion parameters that is subjected to the design process require careful consideration so that precise outcome could be obtained.

2.3 Ductility and Behaviour Factor

Ductility design for earthquake loads is another important consideration. The degree of ductility indicates the extent to which earthquake energy is absorbed by the structure that would otherwise cause it to continue to resonate (Andrew, 2008). It is preferred to ensure that the joints of a structural frame such as beam-column connections be of sufficient ductility to allow movement under earthquake forces, without joint failure (IEM Council, 2005). According to Eurocode 8 (2004) there are three classes of ductility which are ductility class low (DCL), ductility class medium (DCM), and ductility class high (DCH). As for Malaysia, DCL and DCM may be recommended (Chiang and Jeffrey, 2011).

According to Elghazouli (2009), designing structures to remain elastic in large earthquakes is likely to be uneconomic in most cases, as the force demands will be very large. A more economical design can be achieved by accepting some level of damage short of complete collapse, and making use the ductility of the structure to reduce the force demands to acceptable levels. The reduction is accomplished by introducing the behaviour factor, q. The same concepts also exists in American codes which known as force or strength reduction factor, R (ICC, 2006). The value of behaviour factor, q is strongly related to the level of ductility where ductility design corresponds to high value of behaviour factor, q and vice versa (Adiyanto, 2016).

Figure 2.5 shows the equivalence of ductility and behaviour factor with equal elastic and elastic displacement. F_{el} is the peak force that would be developed if it responded to the earthquake elastically, and F_y is the yield load of the system. In Eurocode

8 (2004), the force reduction is simply equal to the ductility as in Figure 2.5 where $q = \frac{F_{el}}{F_y}$

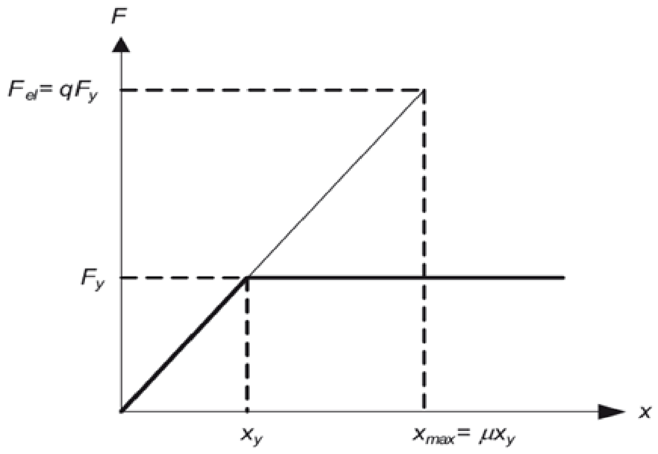

Figure 2.5 Equivalence of ductility with behaviour factor with equal elastic and inelastic displacement

Adiyanto et al. (2014) investigated on the effects of behaviour factor, q on the total cost of material for seismic design of low rise RC hospital building. Based on the research 5 values of behaviour factor, q which are q = 2.3, 3.1, 3.9, 4.7, and 5.5 had been used for DCM. It is assumed to be at moderate seismic region in Malaysia with peak ground acceleration, a_{gR} is equal to 0.12g. It is found that the selection of behaviour factor, q for design plays an important role in influencing the cost. This is due to the final conclusion that has been made which is the total cost of material can be saved up to 22% when using behaviour factor, q = 3.9 compared to the behaviour factor, q = 2.3. However the total cost of material is increasing when the behaviour factor, q > 3.9 due to rapid increasing of the total weight of steel bar for column shear reinforcement.

Ramli et al. (2017) studied on the costing impact to the adoption of seismic action in the designs. The comparison is made in term of total reinforcement requirements between design practice using Eurocode 2 (2004) and similar design including requirements in Eurocode 8 (2004). The analysis and design is based on structure frame element (beams and columns) for five and ten storey buildings. For non-seismic design, it is designed based on Eurocode 2 (2004) and consider only live load and dead load. For ductility class, this study only considered two classes of ductility because DCH is not practical in Malaysia.

In the modal analysis, different ductility class showed different mode shape and time period. The natural modes provided an excellent insight into the behaviour of the structures. The period of the buildings for EC2 design took a longer time compared to buildings with DCL and DCM design. Ductility class design made the total stiffness of the buildings increase with the increase of column design. Larger column size gives more stiffness. Hence it can be concluded that the consideration of ductility class values in analysis and design is very important in designing a seismic building as it will influences the total amount of reinforcement required and the cost of a project.

2.4 Strong Column Weak Beam Design

According to Majid (2017), based on in-situ field observation due to 2015 Ranau earthquake it is clear that the columns experienced significant damage compared to beams. This concept is called as Strong Column – Weak beam design. Strong Column – Weak Beam concept has been proposed for seismic design. (Filiatrault et al., 1998a; Elanshai and Sarno, 2008; Kirtas and Kakaletsis, 2013). Basically, the concept is when the column member is designed stronger than the beam to prevent the failure during strong earthquake so that the reinforced concrete structure can provide the energy dissipation as well as possible. The capacity of column must be greater than the capacity of beam (Elanshai and Sarno, 2008). According to Seah Ivy (2012) the column is designed stronger than the beam to ensure that the column members able to remain elastic to provide stability and strength of the stories above.

The equation from Eurocode 8 was used in strong column weak beam concept. In Eurocode 8 (2004), the no-collapse requirement under the seismic design situation is considered to have been met if the resistance, equilibrium, foundation stability, seismic joints and including the ductility condition are met. In the case of ductility condition, the formation of a soft storey plastic mechanism shall be prevented by satisfying the following condition.

$$\Sigma M_{Rc} \geq \Sigma 1.3\ M_{Rb} \qquad 2.1$$

Where

$\sum M_{Rc}$ is the sum of the design values of the moments of resistance of the columns framing the joint. The minimum value of column moments of resistance within the range of column axial forces produced by the seismic design situation should be used in expression above.

$\sum M_{Rb}$ is the sum of the design values of the moments of resistance of the beams framing the joint. When partial strength connections are used, the moments of resistance of these connections are taken into account in the calculation of $\sum M_{Rb}$.

2.5 Seismograph Network in Malaysia

In Malaysia, there is a national information centre for seismology called as Malaysian Meteorological Service (MMS). MMS provides information, advice and consultation related to seismic activity to users such as the engineers, architects, and planners for socio-economic development of the country. Due to increasing demand for seismological information in Malaysia, MMS has operated an amount of seismic stations around the country. Earthquake acceleration records are obtained from seismographs which record the rapidly changing accelerations or velocities throughout the duration of a quake (Andrew, 2008).

The seismic station are capable to detect and identify earthquake in and around Malaysia. The sensor at different places will detect the seismic waves approaching them and form a set of data of arrival time. Using the data, the source of the wave could be trace back. The stations also will locate the seismic epicentre and its magnitude. Figure 2.6 below shows several seismological stations in Malaysia (Abas, 2001).

Figure 2.6 Seismological stations of Malaysia (Abas, 2001)

2.6 Summary

In summary, seismic design approach for future constructions of the buildings in Malaysia is worthwhile to be considered. This is due to the distant ground motion that have been recorded by Malaysian network of seismic stations coming from neighbouring countries. From the literature review, the selection of the characteristic of seismic design is noted to be very important in analysis and design which includes the PGA and Ductility class. This is because difference value of them will cause an influence to the cost of a project. Therefore, this study will be conducted to understand on the effect of different class of ductility and different value of PGA in seismic design along with its building cost referring to Eurocode 8 as seismic provision.

CHAPTER 3

METHODOLOGY

3.1 Introduction

Sequence of steps in conducting overall design analysis is crucial in achieving the objectives of this study. Hence, this chapter is dedicated to discuss on the steps carried out to determine the influence of both magnitude of PGA and ductility class on the amount of steel reinforcement. In general there are three major phases in overall design process. The summary of research methodology is shown in Figure 3.1. In following section, steps involved in each phase will be discussed in details.

3.2 Summary of Research Methodology

This research is carried out on three phases. The first phase is model generation using Tekla structural software. Second phase is seismic design based on Eurocode 8 (2004) for earthquake resistance. The design was carried out with different parameter of ductility class and PGA value. The outcome will be the amount of steel reinforcement for the member. The final phase is seismic analysis and taking off that were perform after getting the flexural and shear reinforcement design requirement at phase 2. Taking off process was performed on the beam and column elements to calculate total steel reinforcement for seismic design building.

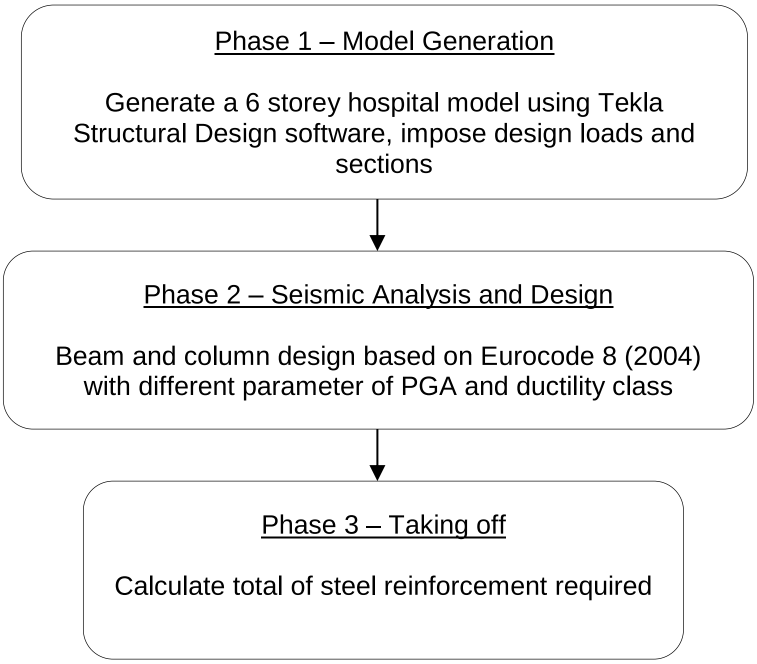

Figure 3.1 Flowchart of seismic design and analysis

3.3 Phase 1 – Model Generation

A six (6) storey hospital building were selected as the main model for this study. All models are design according to Eurocode 8 (2004) using Tekla structural software. The cross sections of the structural members for roof beam and floor beam measured (250x550) and (350x600) mm^2 respectively. Table 3.1 shows the summary of the member cross section. The frame featured three bays, with 1.5 m total span and 3.6 m column height. The concrete strength was assumed to be 30 MPa. Figure 3.2 and Figure 3.3 shows the side and plan view of hospital building model generated in Tekla software.

Table 3.1 Size of members of the frame

Member	**Text**
Roof Beam	250 x 550
Floor Beam	350 x 600
C2	450 x 450
C1	500 x 500

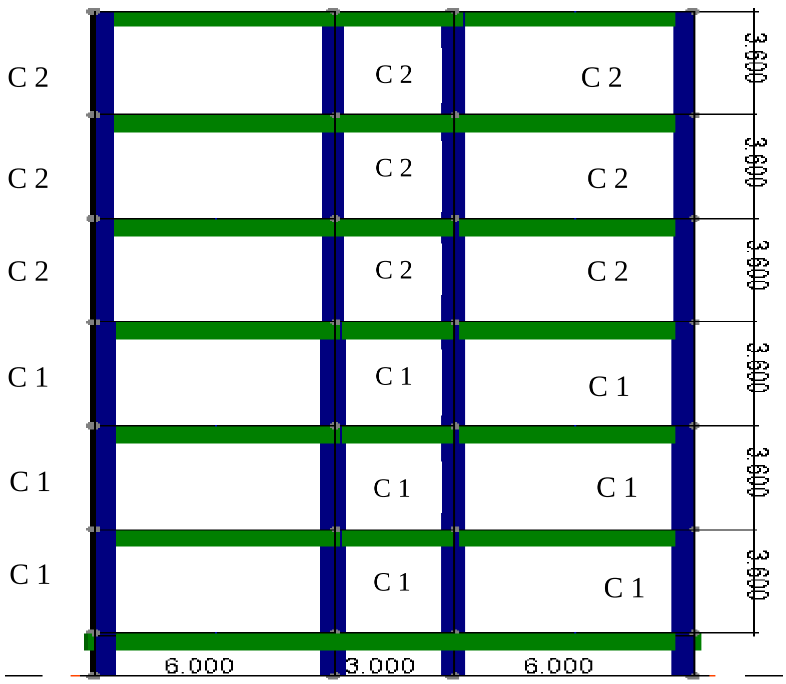

Figure 3.2 Side view of hospital building frame generated in Tekla structural software

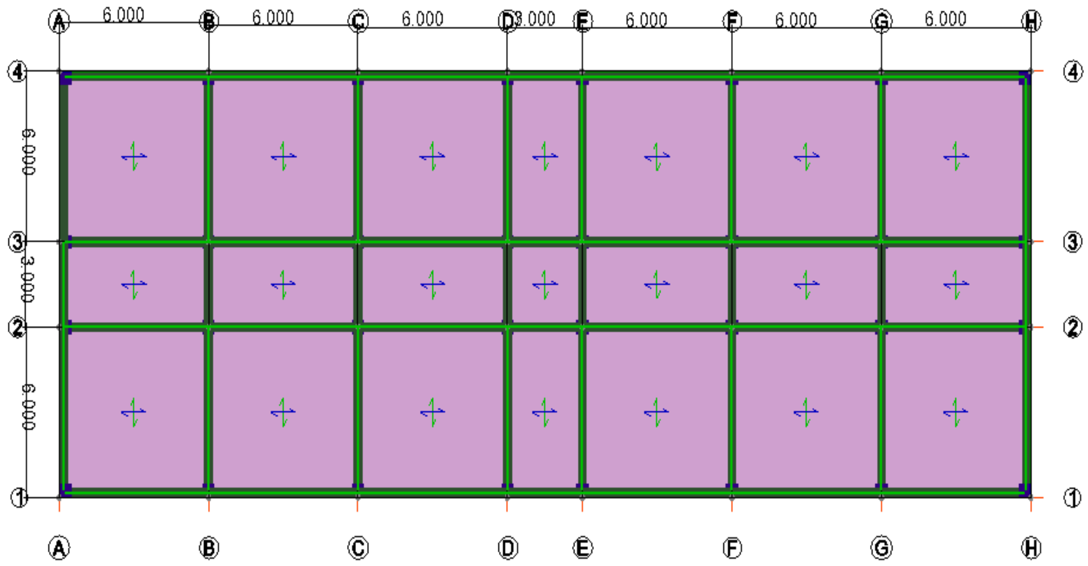

Figure 3.3 Plan view of hospital building

3.4 Phase 2 – Seismic Design

In phase 2, the hospital building is designed based on Eurocode 8 using Tekla software. Beams and columns were design in order to get the total reinforcement required. The various parameter that had been used are complying with the current condition of our country. This study will consider on Soil Type D only which represents the soft soil based on Eurocode 8 (2004). The material properties for the hospital building is shown in Table 3.2 in accordance to Mc Kenzie (2004). The concrete strength is 30 MPa as Miska (2015) said that in designing the structural members for seismic design, the concrete strength should not be less than 20 MPa, and the reinforcements needs to fulfill the requirement stated in provision used.

Table 3.2 Weight of materials (Mc Kenzie, 2004)

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

In this study, hospital building were used and is categorized in Category A for load distribution as stated in Eurocode 1 (2002) shown in Figure 3.3. Therefore, the live load, q_k imposed on the floor and roof of this category will be 2.0 kN/m^2 and 0.4 kN/m^2 respectively. Table 3.4, Table 3.5, and Table 3.6 shows the imposed load on floor, roof categorization and imposed load on roof as stated in Eurocode 1 (2002).

Table 3.3 Categories of building use (Eurocode 1, 2002)

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1])	C1: Areas with tables, etc. e.g. areas in schools, cafes, restaurants, dining halls, reading rooms, receptions. C2: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms C3: Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. C4: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. C5: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sport halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	D1: Areas in general retail shops D2: Areas in department stores

Table 3.4 Imposed loads on floors, balconies and stairs in buildings (Eurocode 1, 2002)

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- Floors	1.5 to 2.0	2.0 to 3.0
- Stairs	2.0 to 4.0	2.0 to 4.0
- Balconies	2.5 to 4.0	2.0 to 3.0
Category B	2.0 to 3.0	1.5 to 4.5
Category C		
- C1	2.0 to 3.0	3.0 to 4.0
- C2	3.0 to 4.0	2.5 to 7.0 (4.0)
- C3	3.0 to 5.0	4.0 to 7.0
- C4	4.5 to 5.0	3.5 to 7.0
- C5	5.0 to 7.5	3.5 to 4.5
Category D		
- D1	4.0 to 5.0	3.5 to 7.0 (4.0)
- D2	4.0 to 5.0	3.5 to 7.0

Table 3.5 Categorization of roofs (Eurocode 1, 2002)

Categories of loaded area	Specific Use
H	Roofs not accessible except for normal maintenance and repair
I	Roofs accessible with occupancy according to categories A to D
K	Roofs accessible for special services, such as helicopter landing areas

Table 3.6 Imposed loads on roofs of category H (Eurocode 1, 2002)

Roof	q_k [kN/m^2]	Q_k [kN]
Category H	q_k	Q_k
NOTE 1 For category H q_k may be selected within range 0.00 kN/m^2 to 1.0 kN/m^2 and Q_k may be selected within the range 0.9 kN to 1.5 kN. Where a range is give the values may be set by the National Annex. The recommended values are: q_k = 0.4 kN/m^2, Q_k = 1.0 kN NOTE 2 q_k may be varied by the National Annex dependent upon the roof slope. NOTE 3 q_k may be assumed to act on an area A which may be set by the National Annex. The recommended value for A is 10 m^2, within the range of zero to the whole area of the roof. NOTE 4 See also 3.3.2 (1)		

3.4.1 Seismic Base Shear Force, F_b

In this study all models will be subjected to the same gravitational load (dead load and imposed load). However the models will be subjected to different lateral load as the parameter of this study which are behaviour factor, q and magnitude of PGA are varies. As proposed in Eurocode 8, the seismic action on building for each horizontal direction in which the building is analysed can be represented by the base shear force, F_b which can be determine using the following expression:

$$F_b = S_d\ (T_1).m.\lambda \qquad 3.1$$

Where $S_d\ (T_1)$ is the ordinate of design response spectrum, m is the total mass of the building, T_1 is the fundamental period of vibration of the building for lateral motion in the direction considered, and λ is the correction factor, λ = 0.85 if T_1 < 2 T_c and the building has more than two storey, or λ = 1.0.

3.4.2 Design Response Spectrum Analysis

To avoid explicit inelastic structural analysis in design, elastic analysis based on a response spectrum reduced with respect to the elastic one is performed. Response spectrum also need to be identified first to get base shear force, F_b. Type of spectra that is used in this study is Type 1 spectra where the surface magnitude is not greater than 5.5 while the ground type used is Soil Type D. Table 3.7 below shows the value parameters describing the recommend Type 1 elastic response spectra.

Table 3.7 Values of the parameters describing the recommended Type 1 elastic response spectra for Soil Type D

Ground Type	S	T_B (s)	T_C (s)	T_D (s)
D	1.35	0.20	0.8	2.0

According to Eurocode 8 (2004), design spectrum for elastic analysis is as following expression:

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

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

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

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

Where

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

a_g is the design ground acceleration

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

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

T_D is the value defining the beginning of the constant displacement response range of the spectrum

S is the soil factor

η is the damping correction with a reference value of η = 1 for 5% viscous damping

3.4.3 Design Ground Acceleration

By referring to Eurocode 8 (2004), design ground acceleration, a_g can be expressed as following expression:

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

Where γ_I is correspond to importance factor and a_{gR} is the reference peak ground acceleration.

The value of the importance factor can be determined by referring to the importance classes for building classification as shown as Table 3.8. In this study, the value of γ_1 for importance class of IV are equal to 1.4. The recommended importance factor is to offer better protection of life for such building due to its importance after disaster (Fardis et. al., 2015).

Table 3.8 Importance classes for buildings (Eurocode 8, 2004)

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, central institutions etc.
IV	Buildings whose integrity during earthquakes is vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

The value of reference peak ground acceleration, a_{gR} is based on PGA for Malaysia. According to MOSTI (2009) and Adnan et. al., (2008), seismic hazard map for Malaysia is shown in Figure 3.4 and Figure 3.5 for Peninsular and Eastern Malaysia. In this study, the value of reference peak ground acceleration, a_{gR} is taken as 0.04g, 0.08g, 0.12g and 0.16g which covers for both at Peninsular and eastern Malaysia. The class of ductility used for seismic design are Ductility Class Medium (DCM) and Ductility Class Low (DCL).

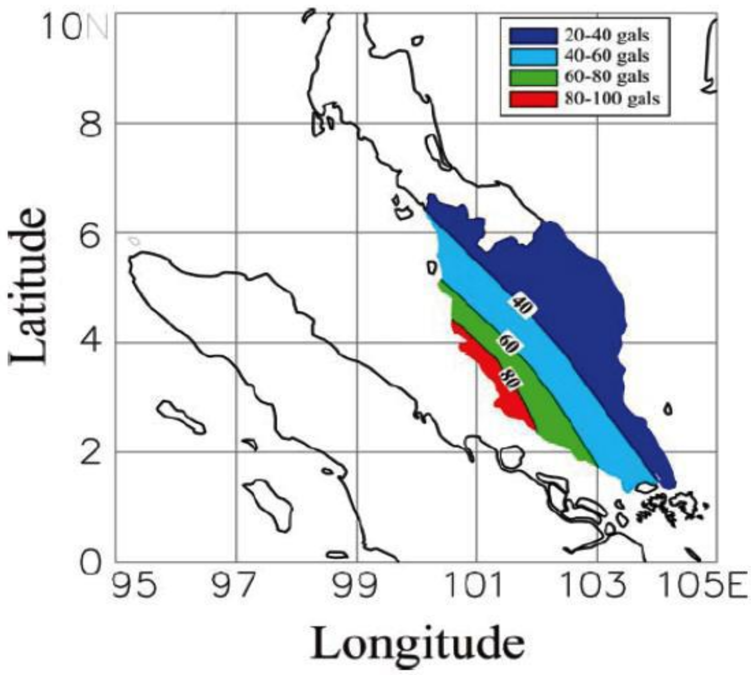

Figure 3.4 Seismic hazard map for Peninsular Malaysia (MOSTI, 2009)

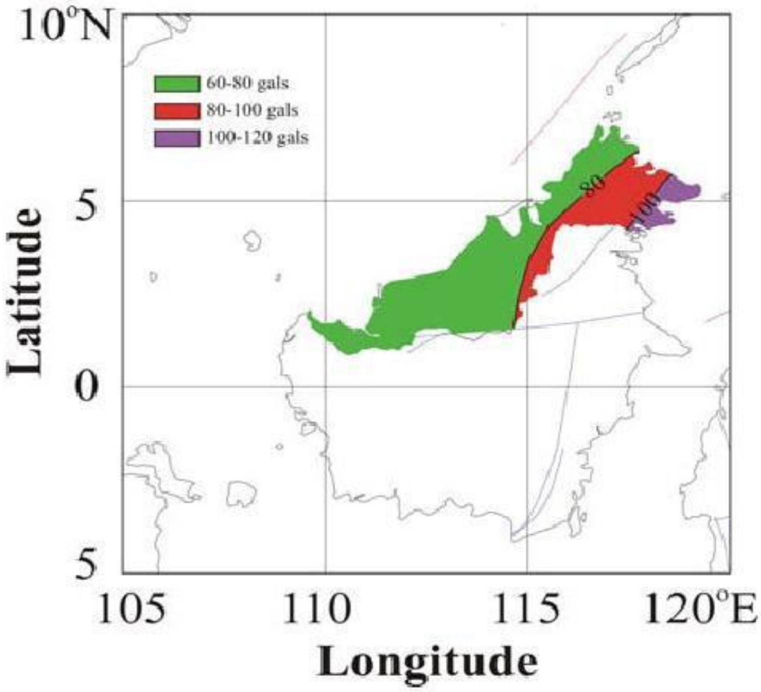

Figure 3.5 Seismic hazard map for Eastern Malaysia (Adnan et. al, 2008)

In order to compare the influence of different value of PGA on the amount of steel reinforcement, four different value of PGA were used which are 0.04g, 0.08g, 0.12g, and 0.16g considering on ductility class medium (DCM). On the other hand, to compare the influence of different class ductility, ductility class low (DCL) and DCM may be recommended for design consideration in Peninsular Malaysia (Chiang et al., 2011). However higher magnitude of PGA cannot be conducted for DCL. This is because the provision in Eurocode has stated that, for low seismicity reference peak ground acceleration, a_{gR} must not exceed 0.08g or product of a_gS does not exceed 0.1g. Hence, reference peak ground acceleration, a_{gR} = 0.04g comply with the provision for both low and medium seismicity and was chosen as fix value in order to determine the effect of different class of ductility to amount of steel reinforcement required. While for medium seismicity no restriction for the design stated in provision.

Hence, for DCL only PGA of 0.04 g is used and for DCM, all levels of PGA are used which are 0.04g, 0.08g, 0.12g, and 0.16g. It is also important to be noted that the value of behaviour factor, q is strongly related to the level of ductility where ductility design corresponds to high value of behaviour factor, q and vice versa. (Adiyanto, 2017). Non-seismic model also has been generated to compare the percentage difference of amount of steel reinforcement for seismic building and non-seismic building. Table 3.9 shows all models of the hospital building that had been considered in this study. Figure 3.6 shows 3D model of the building generated from Tekla structural software.

Table 3.9 All models of the hospital building

Model	**PGA (g)**
Non-seismic	None
DCL – 0.04	0.04
DCM – 0.04	0.04
DCM – 0.08	0.08
DCM – 0.12	0.12
DCM – 0.16	0.16

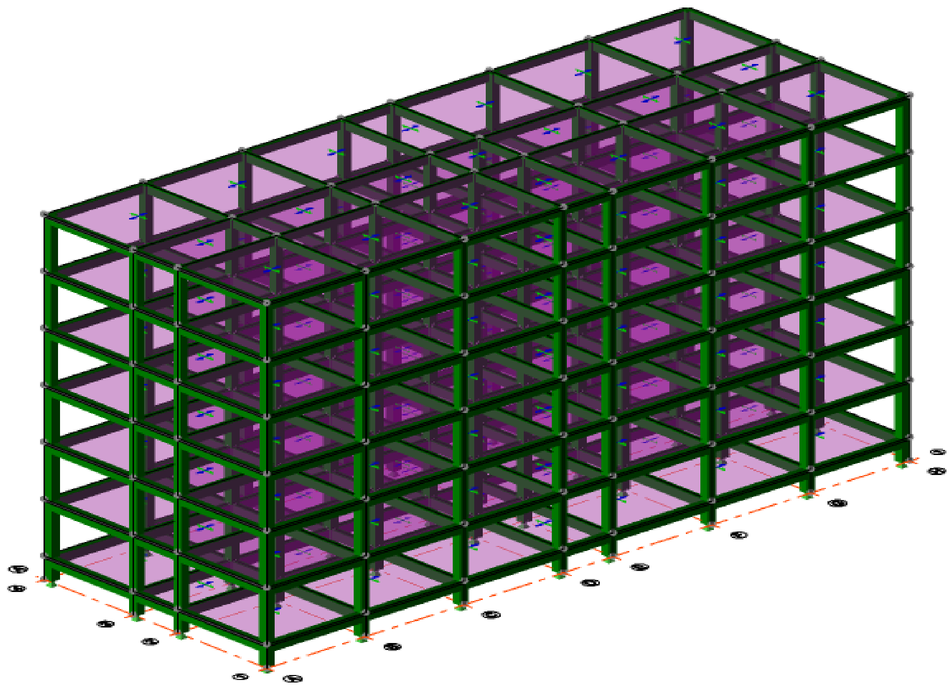

Figure 3.6 3D model of the building generated from Tekla structural software.

3.4.4 Distribution of Lateral Load

According to Eurocode 8, the seismic action effects shall be determined by applying to the two planer models, horizontal forces, F_i to every storey of the building.

$$F_i = F_b \cdot \frac{z_i . m_j}{\sum z_j . m_j} \qquad 3.7$$

Where

F_i is the horizontal force acting on storey, i

F_b is the seismic base shear force

Z_i , Z_j are the height of masses m_i ,m_j above the level of application of the seismic action

m_i , m_j are the storey masses computed

Once the magnitude of base shear force, F_b had been determined, bending moment, shear force and axial load will be obtained from structural analysis. These output will be used for beam and column design.

3.4.5 Beam Design

Beam design was carried out according to Eurocode 8. In this study, the maximum bending moment is chosen as design moment for the analysis. The amount of steel reinforcement proposed will be depending on the maximum bending moment at the section. The higher the bending moment, the higher amount of steel reinforcement required. Figure 3.6 shows the flow chart of beam design to Eurocode 8 (2004).

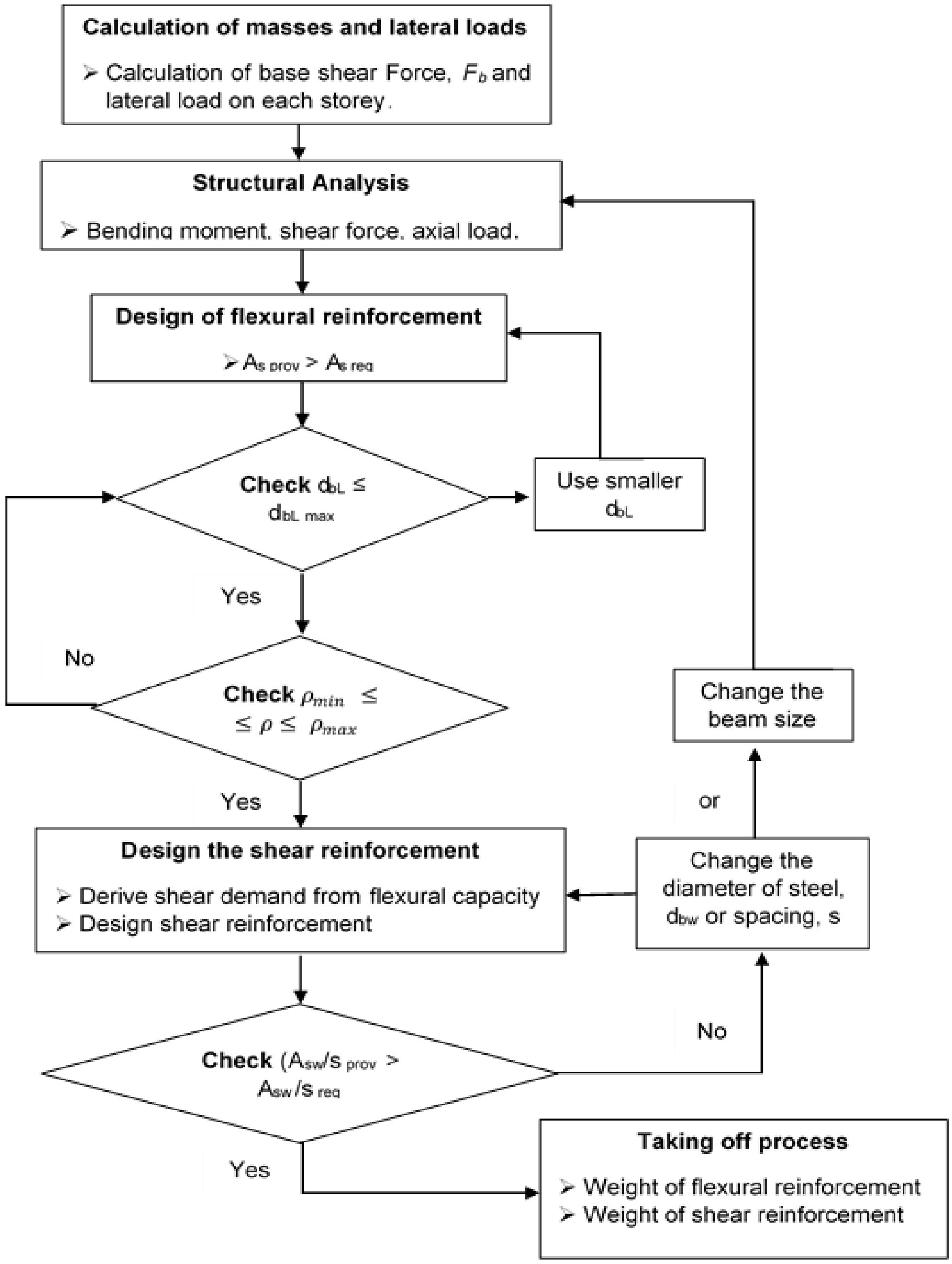

Figure 3.6 Flow chart of beam design according to Eurocode 8 (Adiyanto, 2016)

3.4.6 Column Design

Column design was carried out according to Eurocode 8. Maximum bending moment was used to determine the column size and amount of steel reinforcement needed. Figure 3.7 shows the flow chart of column design to Eurocode 8 (2004).

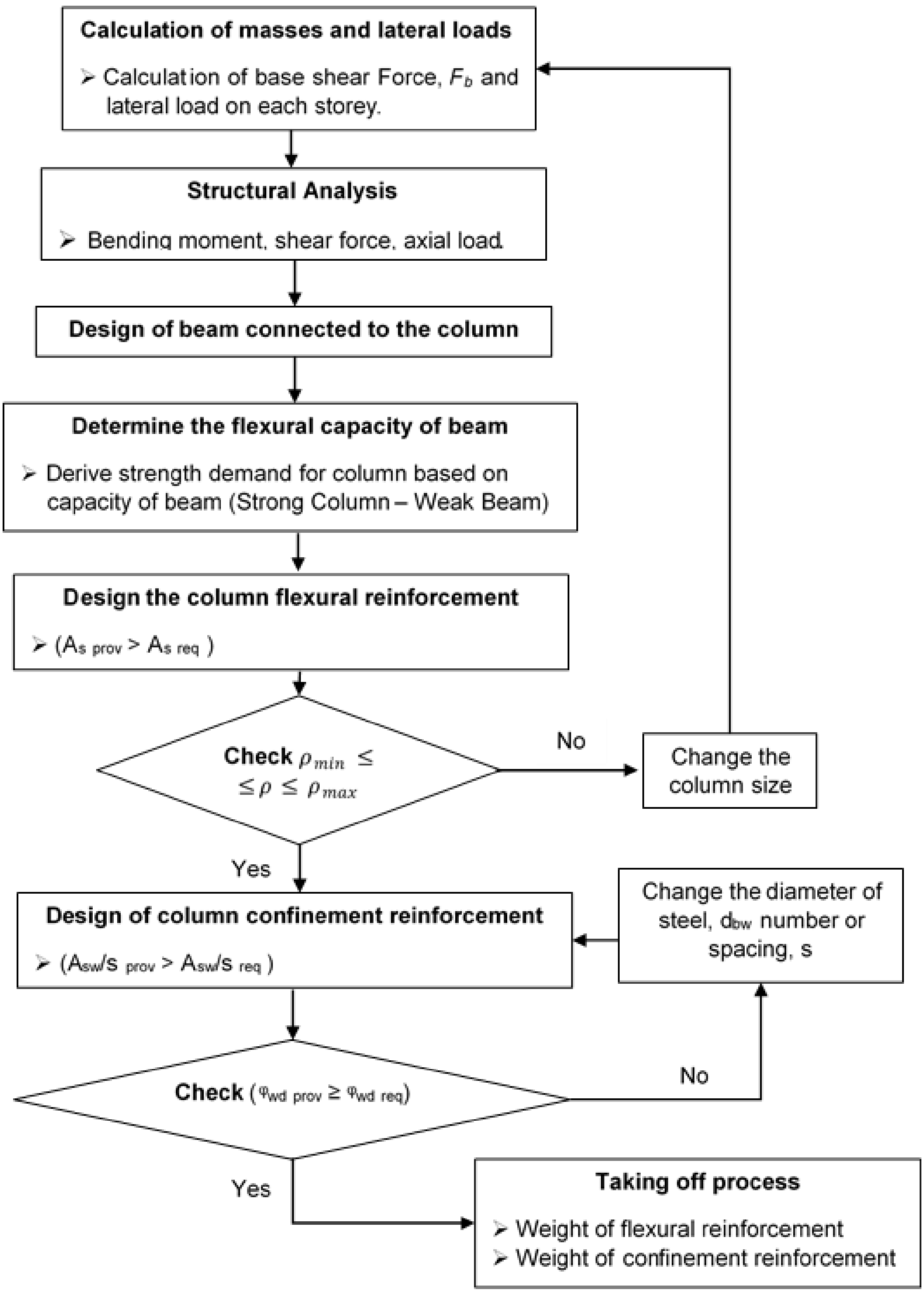

Figure 3.7 Flow chart of column design to Eurocode 8 (Adiyanto, 2016)

3.5 Phase 3 – Seismic Analysis and Taking Off

In the last phase of the research methodology, seismic design on the building frames designed based on various value of reference peak ground acceleration and ductility class was carried out using Tekla structural software. Total mass of the frames was calculated based on the size of the structural member (beam and column) determined in Phase 2. Taking off process will be performed once the flexural and shear reinforcement had satisfied all the design process. Total amount of steel reinforcement required for 1 m^3 of concrete for main and link reinforcement for both beam and column of the buildings will be calculated.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Introduction

This chapter presents the discussion of result obtained in this study. The discussion are discussed in term of comparison of steel required as reinforcement influenced by ductility class and magnitude of PGA. The results obtained will be discussed based on lateral force method as proposed in Eurocode 8 (2004) where the earthquake action on building can be represented by the base shear force, F_b, which will then affect the amount of steel required for the building.

4.2 Design Response Spectrum

Based on equation 3.1 discussed in previous section, it will be used to determine the base shear force of the building which comprises of ordinate of design response spectrum, $S_d (T_1)$ and fundamental period of vibration, T_1. A design response spectrum graph is constructed first to obtain the base shear force, F_b. Then, as proposed in Eurocode 8 (2004) fundamental period of vibration, T_1 of the structure is estimated by using the following equation.

$$T_1 = C_t \, . \left(H^{3/4}\right) \qquad 4.1$$

Where C_t is 0.075 for moment resistance space concrete frames and H is 21.6 metres which is the height of the building, in m, from the foundation or from the top of a rigid basement. Hence, the fundamental period of vibration, T_1 for this structure is 0.75.

As known, in this study all models will be subjected to the same gravitational load (dead load and imposed load). However the models will be subjected to different lateral load as the parameter of this study which are magnitude of peak ground acceleration and ductility class are varies. The value of design response spectrum graph will also varied as the value of parameter varied. This is because the value of reference peak ground acceleration and behaviour factor, q required as in equation 3.2 are varied which will then affect the base shear force, F_b acting on the building.

The first objective of the analysis is to study the effect of PGA on the amount of steel reinforcement required with fix value of behaviour factor, q. The Type 1 design response spectrum has been develop for the analysis and design. Soil Type D, which is soft soil is considered as a building built on this soil type tend to have greater effect on tremor compared to other Soil Type.

Figure 4.1 shows the design response spectrum graph for the discussion. The design response spectrum is develop for behaviour factor, q =3.9 for DCM. Importance factor used is λ_1 = 1.4, for hospital building and reference peak ground acceleration, a_{gR} used are 0.04g, 0.08g, 0.12g and 0.16g.

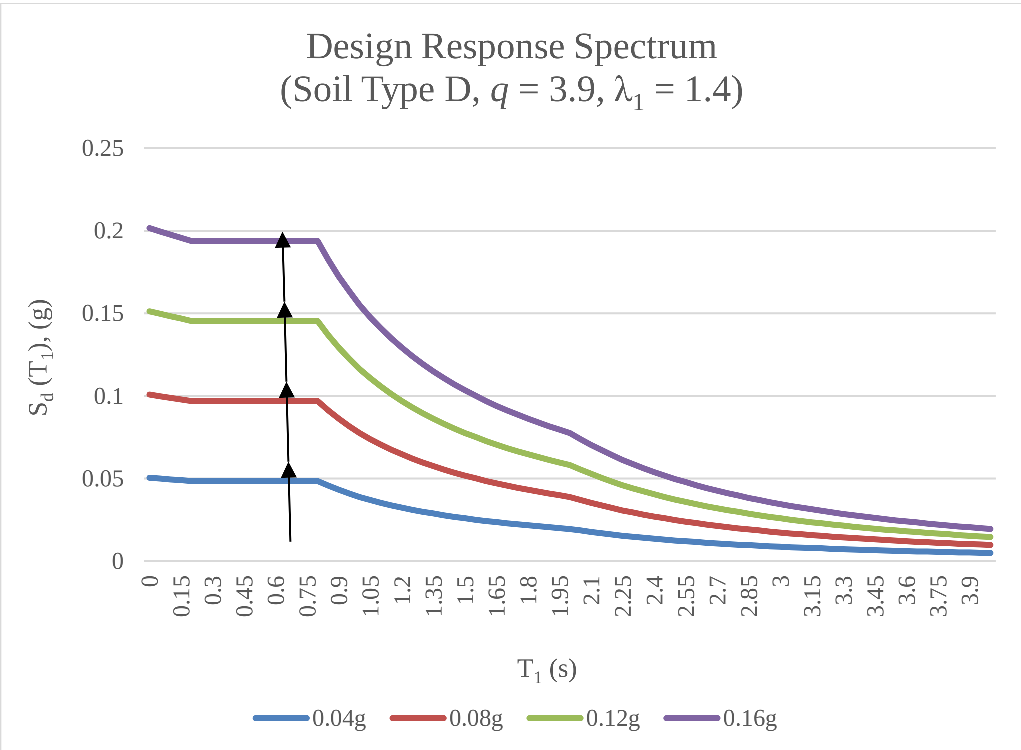

Figure 4.1 Design Response Spectrum for Soil Type D, q = 3.9

From Figure 4.1 it can be seen that for fundamental period of vibration, T_1 of this structure which is 0.75, the value of design response spectrum for increasing magnitude of PGA has become higher. Therefore, it can be concluded that when magnitude of PGA higher, the value of design response spectrum also become higher.

The second objective of the analysis is to study the effect of class of ductility on the amount of steel reinforcement required. The Type 1 design response spectrum has been develop for the analysis and design. Soil Type D, which is soft soil is considered as a building built on this soil type tend to have greater effect to tremor compared to other soil type.

Figure 4.2 shows the design response spectrum for the discussion. The design response spectrum is develop for behaviour factor, q = 1.5 for DCL and q = 3.9 for DCM. Importance factor used is λ_1 = 1.4, for hospital building and the reference peak ground acceleration, a_{gR} = 0.04g.

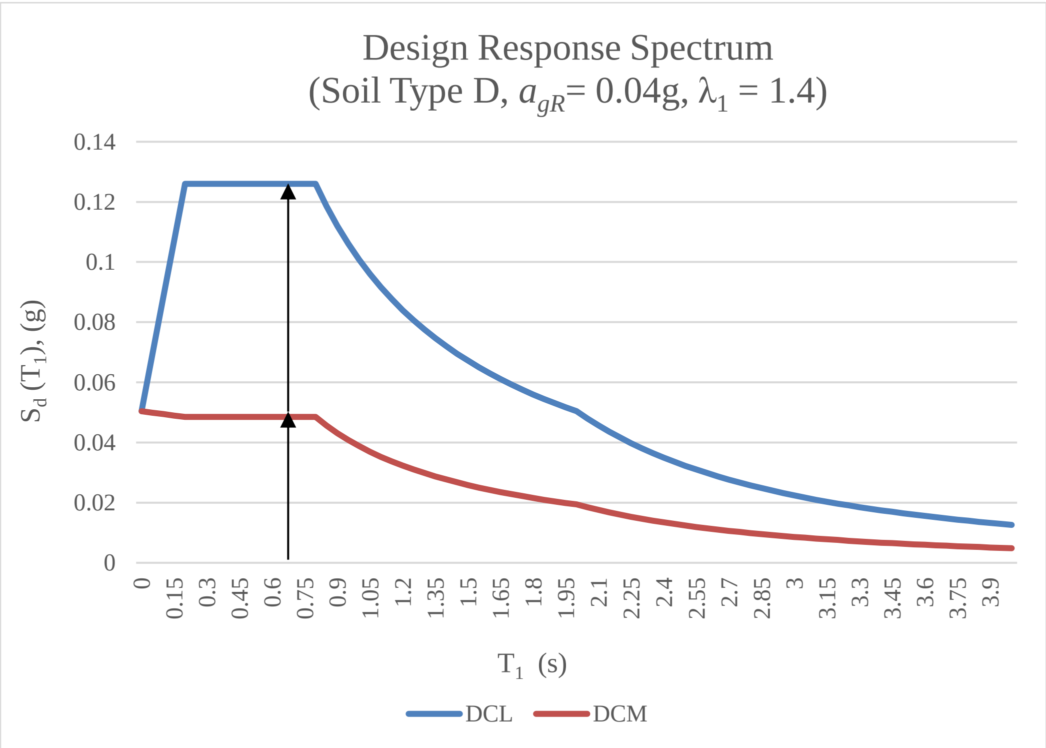

Figure 4.2 Design Response Spectrum for Soil Type D, a_{gR} = 0.04g

From Figure 4.2 it can be seen that for fundamental period of vibration, T_1 of this structure which is 0.75, the value of design response spectrum for lower value of behavior factor, q which is q = 1.5 for DCL is higher compared to behavior factor, q = 3.9 for DCM. Therefore, it can be concluded that when value of behavior factor, q is lower, value of design response spectrum become higher.

4.3 Effect of magnitude of PGA on the amount of steel reinforcement required

In this discussion, the amount of steel reinforcement used is being compared for different magnitude of PGA to non-seismic model. DCM is considered where the behaviour factor, q = 3.9 and considering on Soil Type D. The amount of steel reinforcement used for beam, column and overall reinforcement are discussed in the following section.

4.3.1 Effect of magnitude of PGA on the amount of steel used for beam reinforcement

Figure 4.3 and Figure 4.4 shows the comparison of total weight of steel used for main reinforcement and shear reinforcement respectively for beam 2 (B2). The location of the beam is shown in Appendix I. For main reinforcement, the percentage difference of weight of steel required are increase from 29% to 356% compared to the non-seismic design. For more detail, the increment is 29%, 118%, 245%, and 356% for reference peak ground acceleration, a_{gR} = 0.04g, 0.08g, 0.12g, and 0.16g respectively. While for shear reinforcement, the percentage difference of weight of steel reinforcement increase from 0.8% to 96% compared to non-seismic design. The increment is approximately 1%, 15%, 36%, and 96% for reference peak ground acceleration, a_{gR} = 0.04g, 0.08g, 0.12g, and 0.16g respectively.

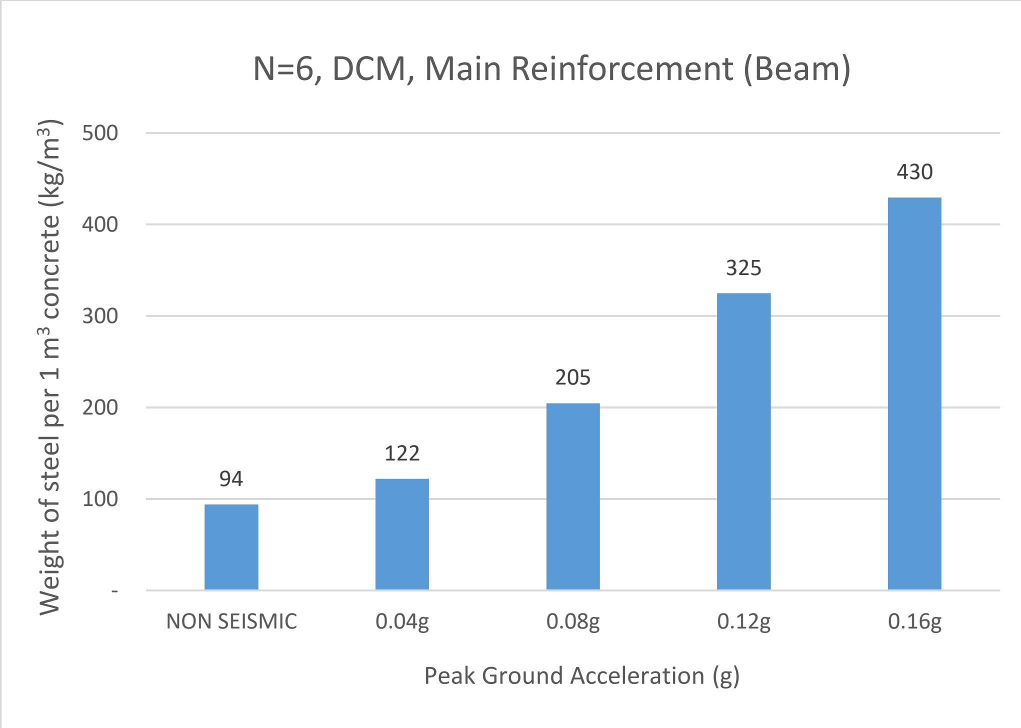

Figure 4.3 Beam main reinforcement for various magnitude of PGA

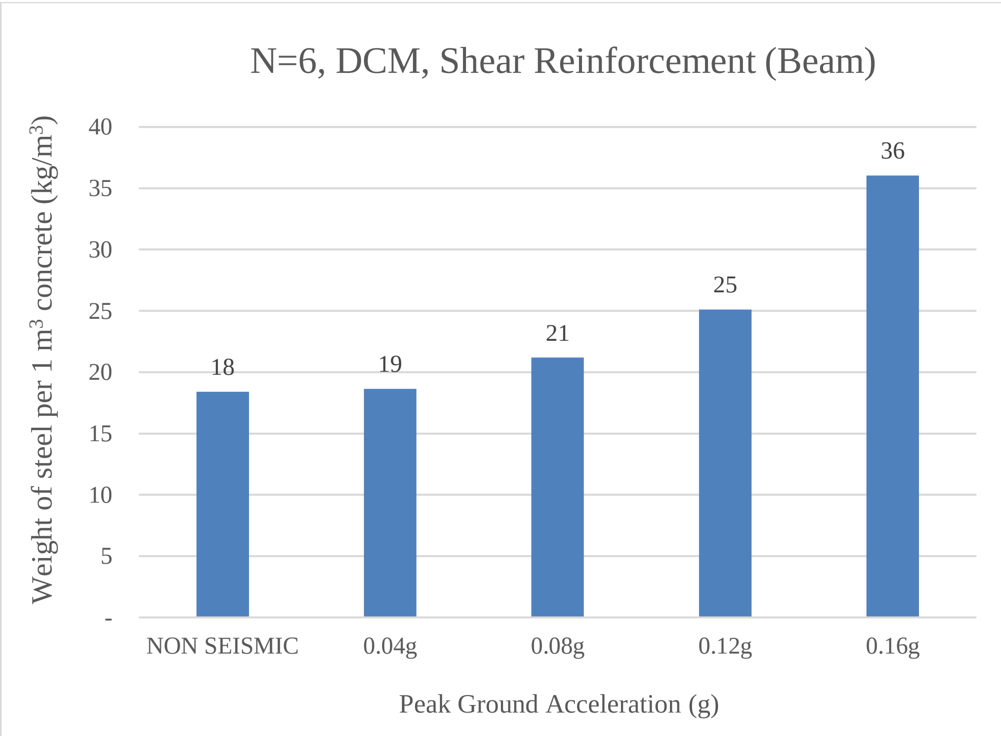

Figure 4.4 Beam shear reinforcement for various magnitude of PGA

For main and shear reinforcement, it can be said that the increment of total weight of steel from the lowest magnitude of PGA is directly related to design response spectrum as discussed in Figure 4.1. It is shown that the value of response spectrum, $S_d\ (T_1)$ for larger magnitude of PGA is higher compared to smaller magnitude of PGA. This is because the higher the magnitude of PGA, the higher the value of response spectrum. Hence, the higher value of response spectrum, $S_d\ (T_1)$ resulted in higher value of base shear force, F_b. When base shear force increase, with fix value of total mass, m and correction factor, λ the bending moment and shear force will be increase and resulting in higher amount of steel reinforcement required for beam. The increment of bending moment and shear force can be seen in Table 4.1. The consistencies of the graph shows that the effect of different magnitude of PGA gives significant effect to the amount of steel reinforcement required for beam. This result is in good agreement with previous finding by Ramli et al. (2017), where greater value of PGA in the research had cause increase in the quantity of reinforcement.

Table 4.1 Parameter for beam design with different magnitude of PGA

Model	Longitudinal bars				Shear reinforcement					
	M_{ed}	$A_{s\ min}$	$A_{s\ req}$	$A_{s\ prov}$	V_{ed}	$V_{rd,c}$	$V_{rd,max}$	$A_{s\ min}$	$A_{s\ req}$	$A_{s\ prov}$
Non-seismic	119	330	523	942	128	77.6	970.4	317	239	574
0.04-M	235	325	1042	1473	132	77.3	966	317	246	574
0.08-M	388	322	1857	2101	186	75.4	933.4	317	358	574
0.12-M	542	316	2283	2945	240	73.3	896.1	317	482	670
0.16-M	656	311	3644	3644	286	70.6	849.7	317	605	804

4.3.2 Effect of magnitude of PGA on the amount of steel used for column reinforcement

Figure 4.5 and Figure 4.6 shows the comparison of total weight of steel used for main reinforcement and shear reinforcement respectively for column 6 (C6). The location of the column is shown in Appendix II. For main reinforcement, the percentage difference of weight of steel provided increase from no changes to 804% compared to the non-seismic design. For more detail the percentage of difference is 367%, 804%, and 647% for reference peak ground acceleration, a_{gR} = 0.08g, 0.12g, and 0.16g respectively and no changes for reference peak ground acceleration, a_{gR} = 0.04g. While for shear reinforcement, the percentage difference of weight of steel reinforcement increase from

0% to 37% compared to non-seismic design. The percentage difference is approximately 0%, 7%, 37%, and 26% for reference peak ground acceleration, a_{gR} = 0.04g, 0.08g, 0.12g, and 0.16g respectively.

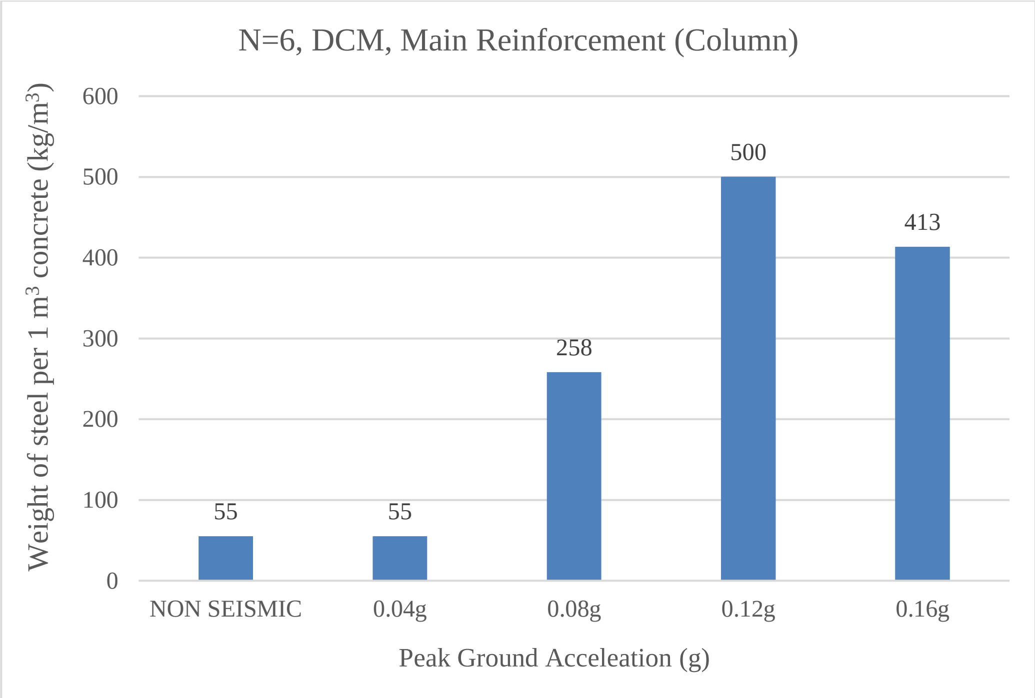

Figure 4.5 Column main reinforcement for various magnitude of PGA

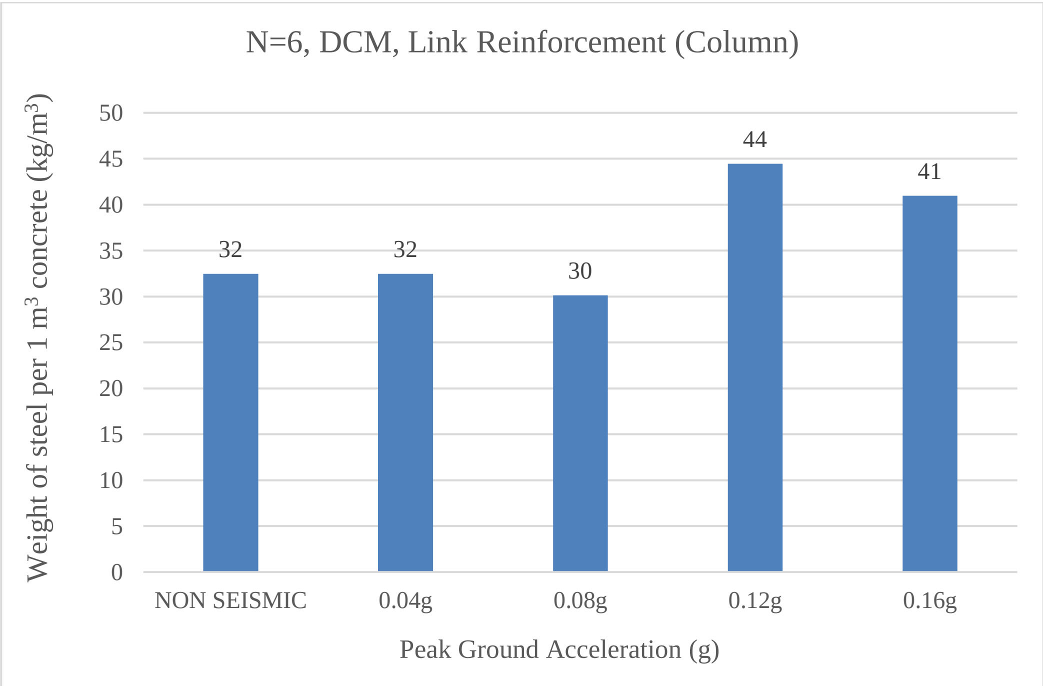

Figure 4.6 Column shear reinforcement for various magnitude of PGA

For main reinforcement, it can be said that increment of total weight of steel from smaller magnitude of PGA to larger magnitude of PGA is directly related to design response spectrum as discussed in Figure 4.1. It is shown that the value of response spectrum, $S_d\,(T_1)$ for larger magnitude of PGA is higher compared to lower magnitude of PGA. This is because the higher value of PGA, the higher the value of response spectrum. Hence, the higher value of response spectrum, $S_d\,(T_1)$ resulted in higher value of base shear force, F_b. When base shear force increase, with fix value of total mass, m and correction factor, λ the bending moment will be increase and resulting in higher amount of steel reinforcement for column. The increment of bending moment of the analysis can be seen in Table 4.2.

Table 4.2 Parameter for column design with different magnitude of PGA

Model	Longitudinal bars							Shear reinforcement	
	M_{res}	M_{ed}	N_{max}	N_{ed}	$A_{s,min}$	$A_{s,max}$	$A_{s,\,prov}$	$A_{s\ req}$	$A_{s\ prov}$
Non-seismic	337	74	5249	1843	1000	10000	1357	0	804
0.04-M	295	201	5249	1843	1000	10000	1357	0	804
0.08-M	431	356	6047	1843	1000	10000	3770	0	698
0.12-M	757	510	7993	1843	1000	10000	9651	0	1005
0.16-M	867	686	9000	1939	1210	12100	9651	0	1005

However, total reinforcement for PGA of 0.16g is slightly divert from expected result where the weight of reinforcement is slightly lower than it supposed to be. This is because the size of the section for the model has been change to a bigger section than the other models due to design failure. Table 4.3 shows the size of members of the frame. As volume of the section is bigger, the reinforcement required become lighter. However, the result of the design is still valid because the greater value of PGA has caused changes in the size of columns which meets the requirements for the increase in the quantity of reinforcement.

Table 4.3 Size of members of the frame for model 0.16-M

Member	**Size of members**
Roof Beam	250 x 550
Floor Beam	350 x 600
C1	550 x 550
C2	550 x 550

While for shear reinforcement, it can be concluded that the weight of steel required for column is insignificant due to the inconsistencies of the graph pattern. Hence, the magnitude of PGA on seismic building has no significant effect to the amount of shear reinforcement required in column.

4.3.3 Effect of magnitude of PGA on overall amount of steel used for reinforcement

Figure 4.7 presents the comparison on total amount of steel reinforcement or the whole building. The trends of the graph is clearer for overall weight of steel reinforcement for beam and column, where the amount of steel reinforcement required increases from lower magnitude of PGA to higher magnitude of PGA. The percentage of difference from non-seismic model is 19% to 330%. For more detail, the increment percentage is 19%, 125%, 272%, and 330% for reference peak ground acceleration, a_{gR} = 0.04g, 0.08g, 0.12g, and 0.16g respectively. Hence, it can be concluded that, magnitude of PGA gives significant effect to overall amount of steel required. The higher value of PGA, the higher amount of steel reinforcement required.

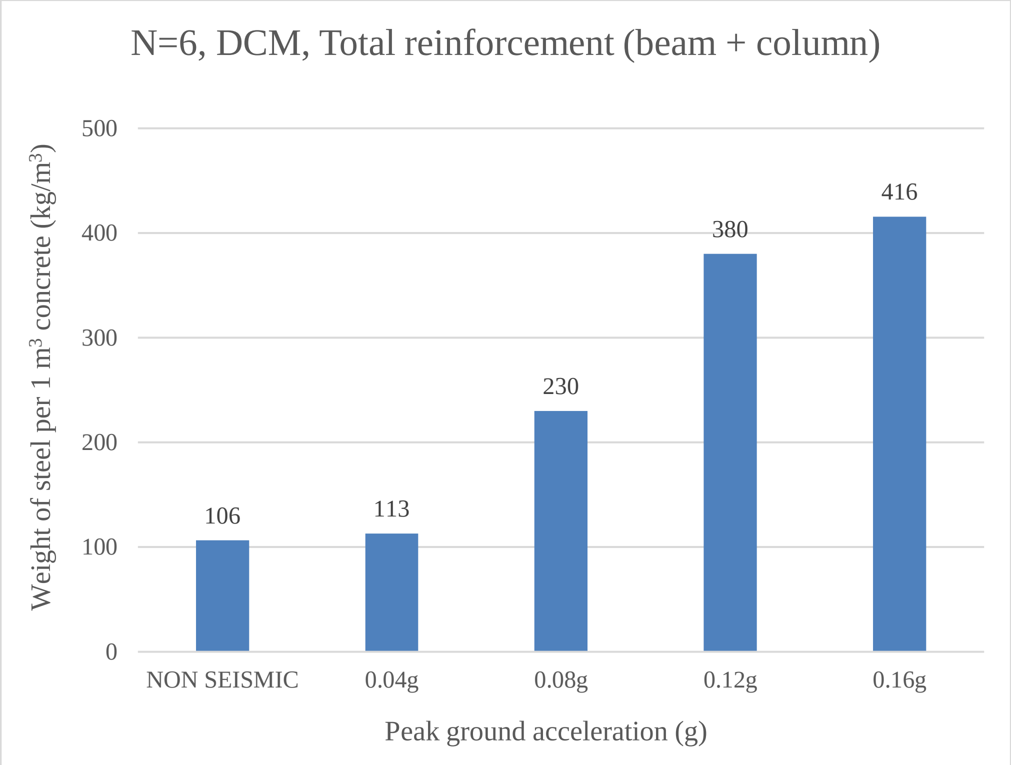

Figure 4.7 Total weight of steel reinforcement per 1m^3 concrete for overall beam and column for different magnitude of PGA

4.4 Effect of class of ductility on the amount of steel reinforcement used

In this discussion, the amount of steel reinforcement used is being compared for different class of ductility with non-seismic model. Fix value of PGA is used which is equal to a_{gR} = 0.04g and considering on Soil Type D. The amount of steel reinforcement used for beam, column and overall reinforcement are discussed in the following section.

4.4.1 Effect of class of ductility on the amount of steel used for beam reinforcement

Figure 4.8 and Figure 4.9 show the comparison of total weight of steel used for main reinforcement and link reinforcement respectively for beam 2 (B2). The location of the beam is shown in Appendix I. For main reinforcement, the percentage difference of weight of steel required increases from 29% to 161% for DCM and DCL respectively compared to the non-seismic design. While for shear reinforcement, the percentage of difference of weight of steel reinforcement increases from 1% to 20% for DCM and DCL respectively compared to the non-seismic design.

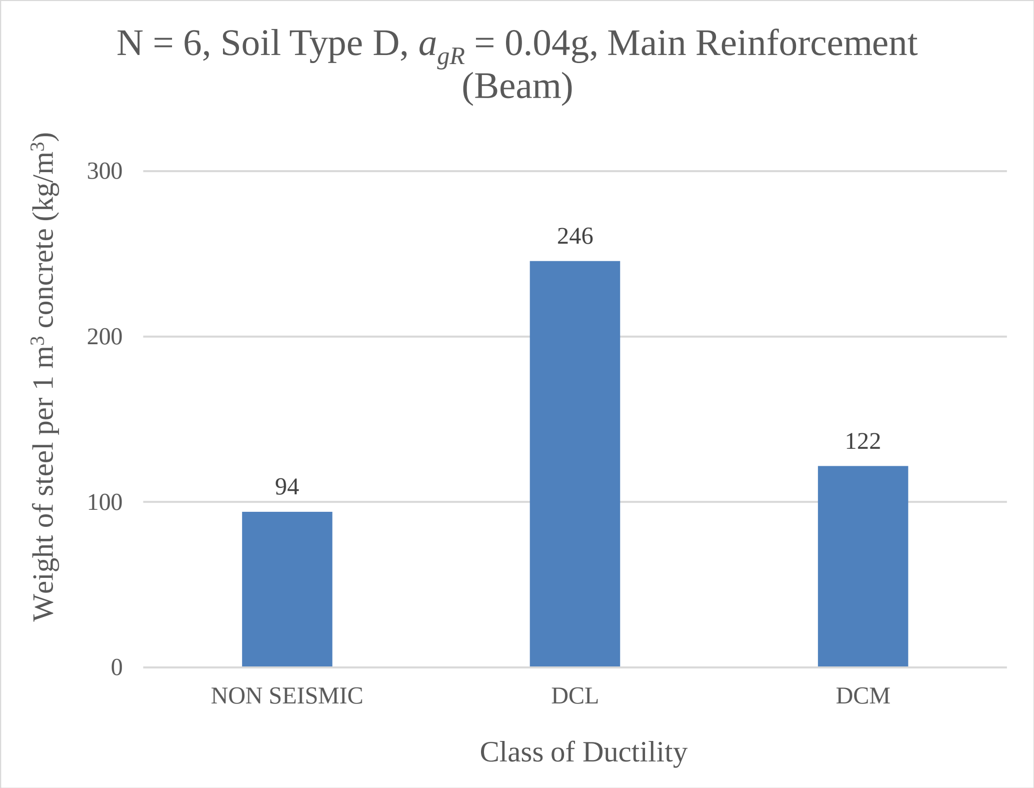

Figure 4.8 Beam main reinforcement for different class of ductility

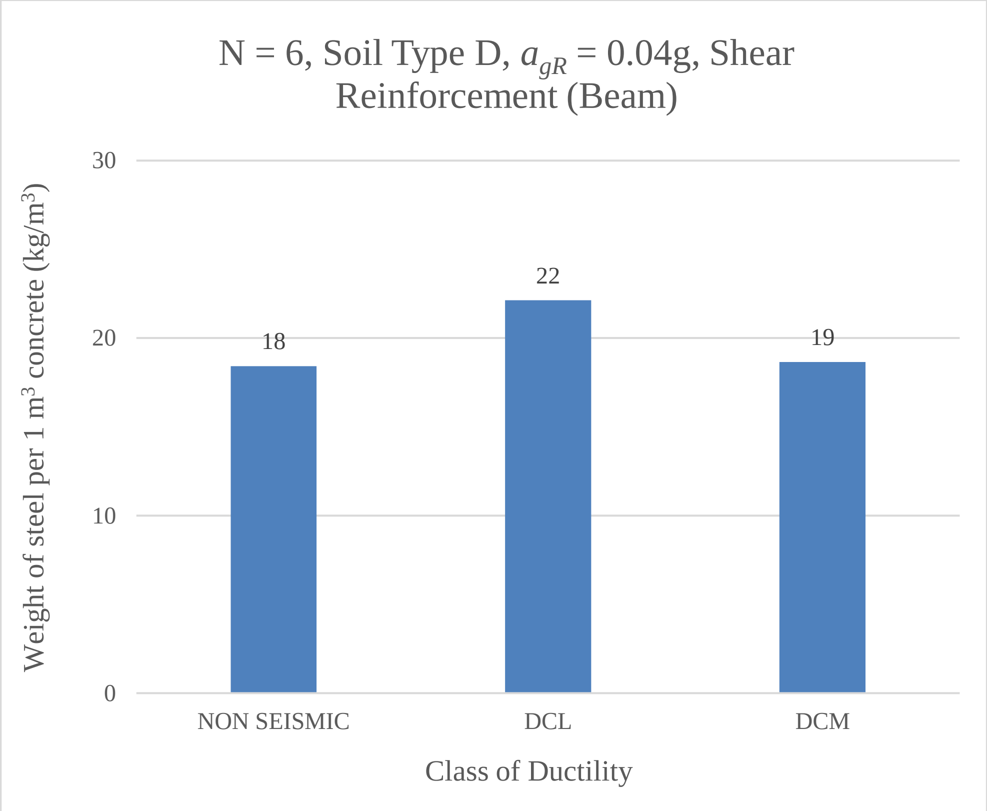

Figure 4.9 Beam shear reinforcement for different class of ductility

For main and shear reinforcement, it can be said that the increment of total weight of steel from DCM to DCL is directly related to design response spectrum shown in Figure 4.2. It shows that the value of response spectrum, $S_d(T_1)$ for DCL is higher compared to DCM. This is because the lower the value of behaviour factor, q the higher the value of response spectrum. Hence, the higher value of response spectrum, $S_d(T_1)$ resulted in higher value of base shear force, F_b. When base shear force increase, with fix value of total mass, m and correction factor, λ the bending moment and shear force will be increase and resulting in higher amount of steel reinforcement for beam. The increment of bending moment and shear force obtained from the analysis and design can be seen in Table 4.4. It can be seen clearly that the magnitude of bending moment and shear force for model with DCL ae the highest compared to DCM and non-seismic. This study is in good agreement with Ramli et al. (2017).

Table 4.4 Parameter for beam design with different class of ductility

Model	Longitudinal bars				Shear reinforcement					
	M_{ed}	$A_{s\ min}$	$A_{s\ req}$	$A_{s\ prov}$	V_{ed}	$V_{rd,c}$	$V_{rd,max}$	$A_{s\ min}$	$A_{s\ req}$	$A_{s\ prov}$
Non-seismic	119	330	523	942	128	77.6	970.4	317	239	574
0.04-M	235	325	1042	1473	132	77.3	966	317	246	574
0.04-L	450	320	2219	2415	208	75	922	317	405	574

4.4.2 Effect of class of ductility on the amount of steel used for column reinforcement

Figure 4.10 and Figure 4.11 show the comparison of total weight of steel used for main reinforcement and link reinforcement respectively for column 6 (C6). The location of the column is shown in Appendix II.

For main reinforcement, the total weight of steel for DCL is higher compared to DCM. For DCL, the percentage difference of weight of steel required compared to non-seismic design is 413 %. While for DCM, weight of steel required is similar with non-seismic design.

While for shear reinforcement, the total weight of steel for DCL is slightly lower than DCM. For DCL, the percentage difference of weight of steel required compared to non-seismic design is 3% lower. While for DCM, weight of steel required is similar with non-seismic design.

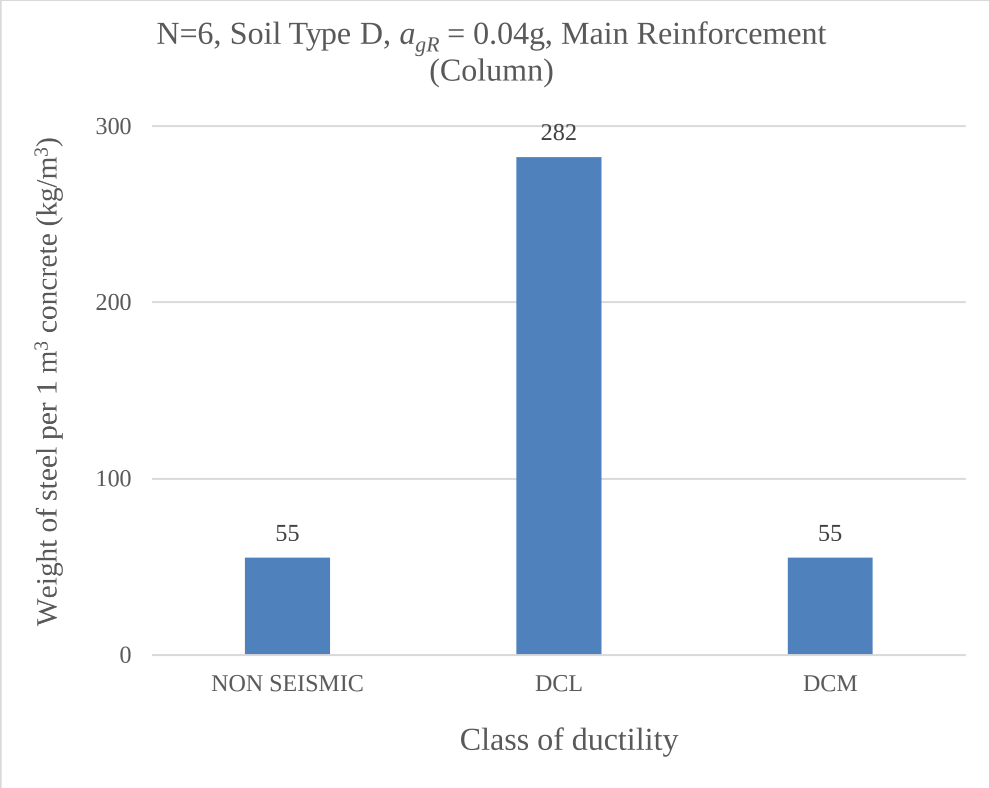

Figure 4.10 Column main reinforcement for different class of ductility

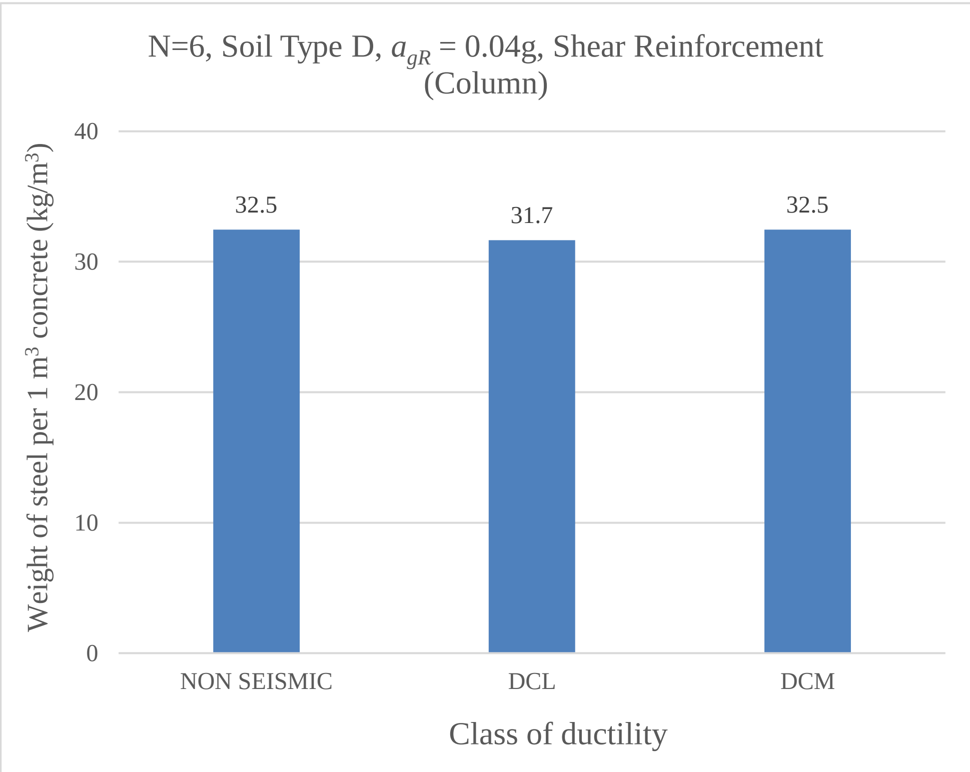

Figure 4.11 Column shear reinforcement for different class of ductility

For main reinforcement, it can be said that higher value of total weight of steel for DCL is directly related to design response spectrum discussed as in Figure 4.2. It is shown that the value of response spectrum, $S_d(T_1)$ for DCL is higher compared to DCM. This is because the lower the value of behaviour factor, q the higher the value of response spectrum. Hence, the higher value of response spectrum, $S_d(T_1)$ resulted in higher value of base shear force, F_b. When base shear force, F_b. increase, with fix value of total mass, m and correction factor, λ the bending moment is increase and resulting in higher amount of steel reinforcement for column. The increment of bending moment from the analysis and design can be seen in Table 4.5.

While for shear reinforcement, it can be concluded that the weight of steel required for column is almost the same for both cases of DCM and DCL. This is because the percentage difference between all models is too small which is from no changes to 3%. Hence, the class of ductility gives no significant effect to the amount of shear reinforcement required.

Table 4.5 Parameter for column design with different class of ductility

Model	Longitudinal bars							Shear reinforcement	
	M_{res}	M_{ed}	N_{max}	N_{ed}	$A_{s,min}$	$A_{s,max}$	$A_{s,prov}$	$A_{s\ req}$	$A_{s\ prov}$
Non-seismic	337	74	5249	1843	1000	10000	1357	0	804
0.04-M	295	201	5249	1843	1000	10000	1357	0	804
0.04-L	554	417	6749	1843	1000	10000	5890	0	698

4.4.3 Effect of class of ductility on overall amount of steel used for reinforcement

Figure 4.12 presents the comparison on total amount of steel reinforcement for the whole building. The trends of the graph is clearer for overall weight of steel reinforcement for beam and column, where the amount of steel reinforcement required increases from DCM to DCL model. The percentage of difference compared to non-seismic model is 6% to 145% for DCM and DCL respectively. Hence, it can be concluded that, class of ductility in seismic design gives significant effect to overall amount of steel required. The lower value of behaviour factor, q, the higher amount of steel reinforcement required.

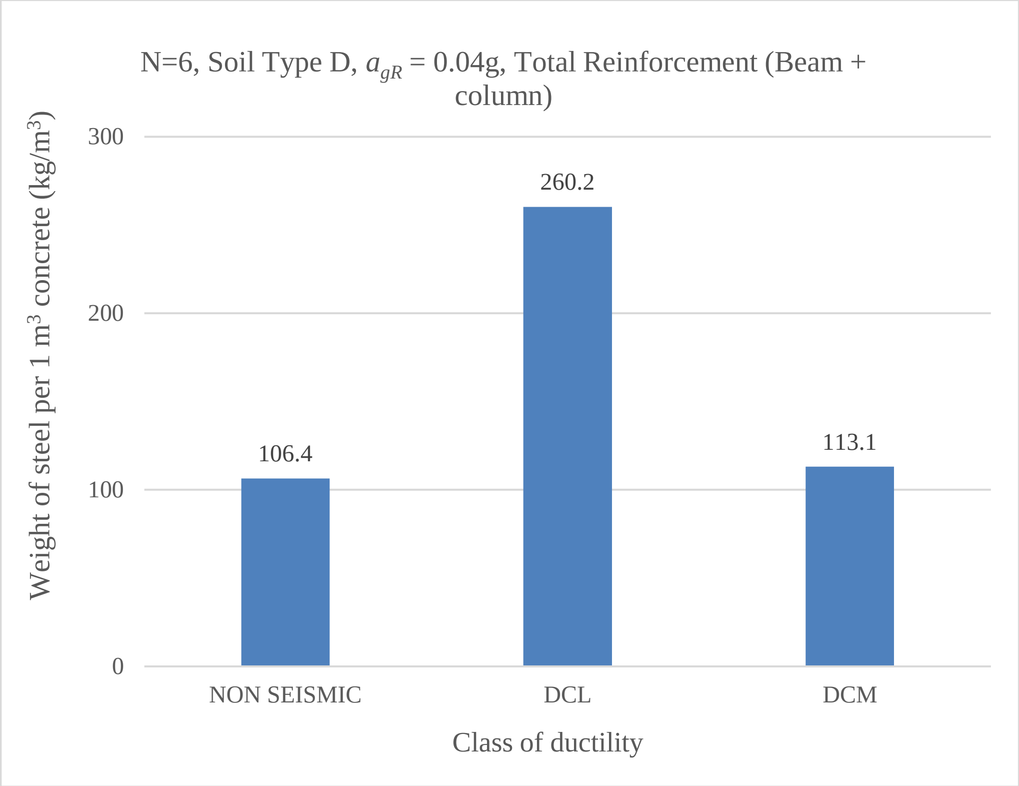

Figure 4.12 Total weight of steel reinforcement per 1 m^3 concrete for overall beam and column for different class of ductility

CHAPTER 5

CONCLUSION

5.1 Conclusions

The objective of this study is to study the influences of PGA and class of ductility on the amount of steel reinforcement required. The amount of reinforcement required is evaluated for beam and column element in a building. To achieve this objectives, a 6 storey RC hospital building has been considered. The model is assumed to be constructed on Soil Type D with compressive strength of concrete, f_{cu} equal to 30 N/mm^2. To compare the percentage increment of steel reinforcement required, a non-seismic model has also been generated with similar fix variables as used for seismic design analysis. Four different magnitude of PGA has been used which are 0.04g, 0.08g, 0.12g, and 0.16g has been design based on DCM. While model of PGA equal to 0.04g has been designed for DCL. The conclusion reached from this study are listed as follow.

- Total amount of reinforcement required in a building is higher when it is subjected to higher magnitude of PGA. The percentage of difference from non-seismic model is 19% to 330%. For more detail, the increment percentage is 19%, 125%, 272%, and 330% for reference peak ground acceleration, a_{gR} = 0.04g, 0.08g, 0.12g, and 0.16g respectively. This is because higher magnitude of PGA resulted in higher value of response spectrum, $S_d\ (T_1)$ which will increase the value of base shear force, F_b. When base shear force increase, the total amount of steel reinforcement required will increase.

- Total amount of reinforcement required in a building is higher when it is subjected to low class of ductility. The percentage of difference compared to non-seismic model is 6% to 145% for DCM and DCL respectively. This is because the lower class of ductility, or lower the behaviour factor, q will resulted in higher value of response spectrum, $S_d(T_1)$ which will increase the value of base shear force, F_b. When base shear force increase, the amount of steel required also will increase.

5.2 Future Recommendation

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

i. This study has examined RC hospital building in the analysis which is categorized as important class IV. Other important class category should also be taken into consideration to study the influences on various type of building.

ii. Since this study has consider Soil Type D in the analysis which is the most critical soil type, similar study on Soil Type A can be conducted, so that the result on the ductility class low can be generated for various magnitude of Peak Ground Acceleration. Hence the percentage different of total amount of steel reinforcement based on ductility class low for various level of PGA can be observe clearer.

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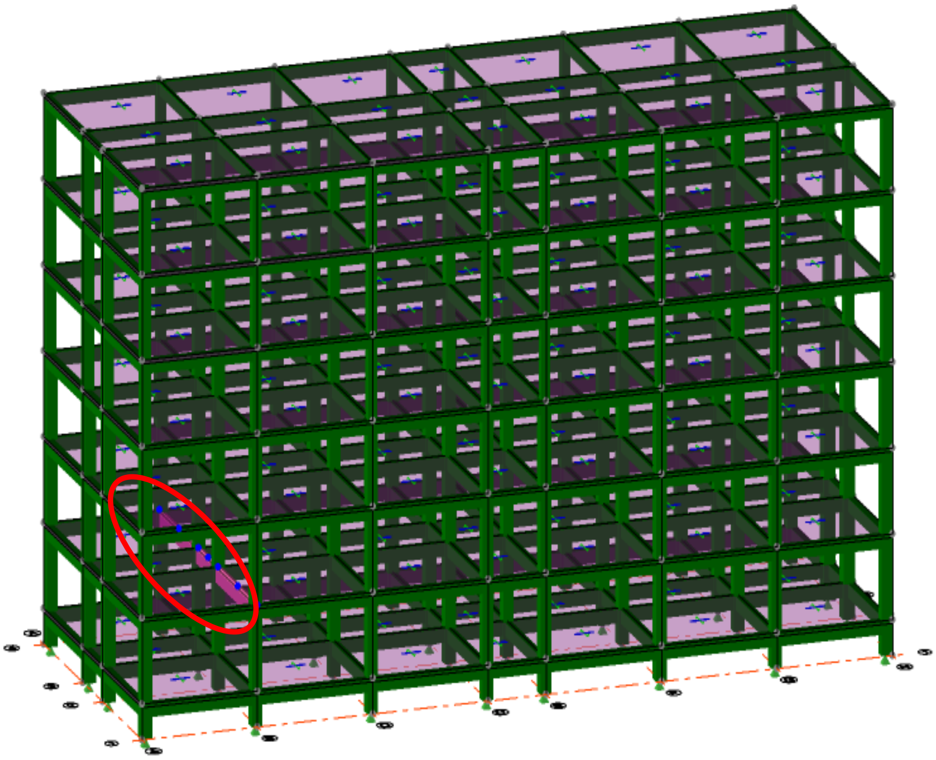
APPENDIX I
BEAM (B2)



APPENDIX II

COLUMN (C6)