

SEISMIC DESIGN OF TWO STOREY REINFORCED CONCRETE BUILDING IN MALAYSIA WITH LOW CLASS DUCTILITY

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Abstract

Since Malaysia is not located in active seismic fault zones, majority of buildings in Malaysia had been designed according to BS8110, which not specify any seismic provision. After experienced several tremors originating from neighbouring countries especially from Sumatra, Indonesia, the Malaysian start to ask questions on integrity of existing structures in Malaysia to withstand the earthquake load. The question also arises regarding the economical effect in term of cost of construction if seismic design has to be implemented in Malaysian construction industry. If the cost is increasing, how much the increment and is it affordable? This paper investigated the difference of steel reinforcement and concrete volume required when seismic provision is considered in reinforced concrete design of 2 storey general office building. The regular office building which designed based on BS8110 had been redesigned according to Eurocode 2 with various level of reference peak ground acceleration, a_{gR} reflecting Malaysian seismic hazard for ductility class low. Then, the all frames had been evaluated using a total of 800 nonlinear time history analyses considering single and repeated earthquakes to simulate the real earthquake event. It is observed that the level of reference peak ground acceleration, a_{gR} and behaviour factor, q strongly influence the increment of total cost. For 2 storey RC buildings built on Soil Type D with seismic consideration, the total cost of material is expected to increase around 6 to 270 %, depend on seismic region. In term of seismic performance, the repeated earthquake tends to cause increasing in interstorey drift ratio around 8 to 29% higher compared to single earthquake.

Keywords: Reinforced concrete, Interstorey drift ratio, Behaviour factor, Eurocode 2, Seismic design.

Nomenclatures

As. Prov.	Area of steel provided, mm ²
a_{gR}	Reference peak ground acceleration, g
F_b	Base shear force, kN
f_{cu}	Concrete compressive strength, N/mm ²
f_y	Yield strength of steel reinforcement, N/mm ²
G_k	Dead load, kN/m
m	Total masses, kg
Q_k	Live load, kN/m
q	Behaviour factor
$Sd(T_1)$	Spectral acceleration at fundamental period of vibration, T_1 , g
T_1	Fundamental period of vibration, sec
V_s	Shear wave velocity, m/s

Greek Symbols

λ	Correction factor
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Abbreviations

DCL	Ductility class low
DCM	Ductility class medium
FB	Floor beam
IDR	Interstorey drift ratio, %
PGA	Peak ground acceleration, g
RB	Roof beam
RC	Reinforced concrete

1. Introduction

Malaysia is situated relatively far away from active seismic fault zone. However, it is clear that the nation is surrounded by high seismicity areas at the west, south, and east part as shown in Fig. 1 [1]. This is associated with the subduction zones between the Indo-Australian plate and Eurasian plate at the west and south part, also the subduction zones between the Eurasian and Philippines plate at the east region. Back to history, before entering the 21st century, Malaysian citizen are not totally aware of the earthquake hazard. They might hear about the catastrophic of 1996 Kobe earthquake in Japan and also the 1999 Koacaeli earthquake in Turkey, and then expressed their sympathy to the victims. After several days they forgot about the disaster and continue their business as usual. However, a large earthquake on 2004 Boxing Day which occurred west of Aceh, in Sumatra, Indonesia had become a wakeup call to all Malaysian as they felt the tremor in their home ground. The earthquake with magnitude M_w 9.0 also generated a disastrous Indian Ocean tsunami with high 'tidal' wave that struck the coast of several countries in Asian region. In Peninsular Malaysia, a total of 76 persons have been reported killed and many properties had been destroyed when the tsunami hit along the northwest coastal areas of Perlis, Kedah, Penang, and to some part of Perak [2]. Then, the tremors also had been felt in Malaysia due to earthquakes with magnitude M_w 8.6 which occurred on 28 March 2005 in Nias and 11 April 2012 in Aceh, Sumatra, Indonesia.

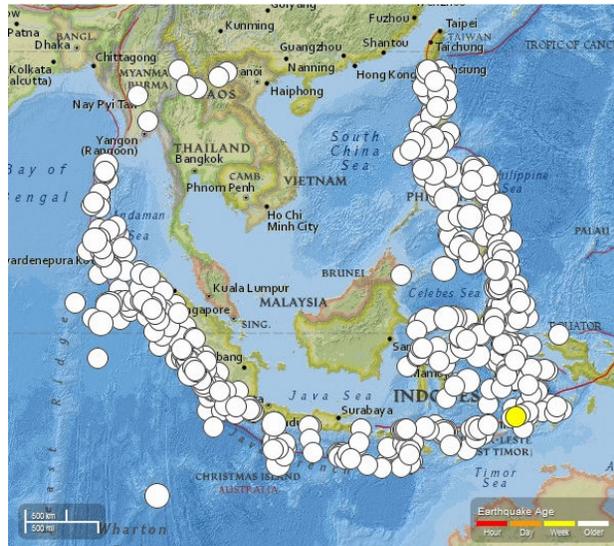


Fig. 1. Earthquake events ($\geq M6.0$) to a depth of 50 km since 1972 [1].

Since Malaysia is not located in active seismic fault zones, majority of buildings in Malaysia had been designed according to BS8110 [3] which not specify any seismic provision. After experienced several tremors originating from neighbouring countries, the Malaysian start to ask questions on integrity of existing structures in Malaysia to withstand the earthquake load. Based on previous investigation [2], it had been reported that most the buildings were in good condition in Peninsular Malaysia and at least 50% of selected buildings were found to experience concrete deterioration problems due to vibration during earthquake. It is also been reported that the vertical element design provision were inadequate for at least 50% of the building evaluated. Then, the Malaysian Public Work Department (JKR) suggested that it was worthwhile to consider seismic design input for new buildings located in medium-to-high risk earthquake zones. Now, the question is arises regarding the economical effect in term of cost of construction if seismic design has to be implemented in Malaysian construction industry. If the cost is increasing, how much the increment and is it affordable?

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 [4]. Therefore, 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 [5]. 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 [6] and FEMA 368 [7] only suggest to considering single earthquake

in analyses. Either in designing the new building or evaluating the existing one, this recommendation had been practised 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 [8]. Recently, it is also reported that repeated earthquake induced 1.3 to 1.4 times increment in maximum storey ductility demand compared to the single one [9]. Therefore, the traditional seismic design procedure which is based on single earthquake should be generally reconsidered [4, 5].

This paper investigated the difference of steel reinforcement and concrete required when seismic provision is considered in reinforced concrete (RC) design of general office building. The original two storey regular office building which designed based on BS8110 [3] had been redesigned according to Eurocode 2 [10] with various level of reference peak ground acceleration, a_{gR} reflecting Malaysian seismic hazard for ductility class low (DCL). Then, the original and newly designed frames had been evaluated using nonlinear time history analysis considering single and repeated earthquakes to simulate the real earthquake event.

2. Analysis Procedure

In this paper, 2 dimensional (2D) analyses had been conducted on typical frame of two storey RC building. The frame was assumed to be designed for general office building with three equal bays of 5.0 m and typical storey height of 3.6 m as shown in Fig. 2. First of all, the generic frame was designed according to BS 8110 [3] to represent the current practice of RC design in Malaysia. The frame was designed with minimum requirement which is just to pass the demand from gravity load. Due to lower magnitude of load, the design of roof beam, RB located at top storey of the frame is differ compared to the design of floor beam, FB at first storey. Typical column design had been used in all storey. Then, the similar frame also had been designed without considering seismic load based on Eurocode 2 [10].

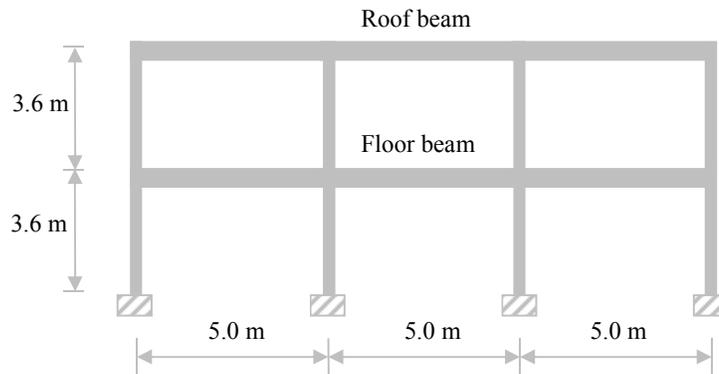


Fig. 2. Elevation of Regular RC Frame Model.

Since only the DCL had been considered in this study for seismic design, the behaviour factor, q used is equal to 1.5 as proposed in Eurocode 8 [6]. For comparison of cost, the frame also had been designed based on elastic response

spectrum where the value of behaviour factor, q is equal to 1. By referring to the seismic hazard maps of Malaysia [2, 11], the value of PGA for Peninsular Malaysia is in range of 0.02 g to 0.10 g for 500 years return period. For the same return period, the value of PGA for East Malaysia (Sabah and Sarawak) is in range of 0.06 g to 0.12 g, which is higher to the east. To cover these wide ranges of PGA for Malaysia, 3 values of PGA had been selected as reference peak ground acceleration, a_{gR} which is equal to 0.02 g, 0.06 g, and 0.12 g. Table 1 depicts all 8 frames used in this study and their design consideration. Only frame labelled as N2BS had been design based on BS 8110 [3] while the rest of it referring to Eurocode 2 [10]. In Table 1, q1.0 and q1.5 correspond to behaviour factor, q equal to 1.0 and 1.5, respectively. P1, P2, and P3 are referring to reference peak ground acceleration, a_{gR} which is equal to 0.02 g, 0.06 g, and 0.12 g, respectively. For example, model labelled with N2q1.0-P2 represents the two storey model (N2) which designed for behaviour factor, q equal to 1 (q1.0) and at reference peak ground acceleration, a_{gR} equal to 0.06 g (P2). In this work, the concrete grade C30 with compressive strength, $f_{cu} = 30 \text{ N/mm}^2$ had been implemented in design for all frames. The yield strength, f_y of steel reinforcement for longitudinal bar and shear for frame N2BS is equal to 460 N/mm^2 and 250 N/mm^2 , respectively as practiced based on BS8110 [3]. Since Eurocode 2 [10] only allows ribbed bars to be used in RC design, the yield strength, f_y for both longitudinal and shear reinforcement is equal to 460 N/mm^2 . General difference for design using both codes [3, 10] is the gravitational load combination. When using BS8110 [3] for design, the gravitational load combination is equal to $1.4G_k + 1.6Q_k$, while for Eurocode 2 [10] is equal to $1.35G_k + 1.5Q_k$. G_k and Q_k correspond to dead and live load, respectively.

Table 1. List of Frames Used and Design Consideration.

No.	1	2	3	4	5	6	7	8
Frame	N2BS	N2EC2	N2 q1.0-P1	N2 q1.5-P1	N2 q1.0-P2	N2 q1.5-P2	N2 q1.0-P3	N2 q1.5-P3
Behaviour factor, q	Not Applicable		1.0	1.5	1.0	1.5	1.0	1.5
a_{gR}	Not Applicable		0.02 g		0.06 g		0.12 g	

In this paper, the lateral force method of analysis [6] had been conducted on all frames which considering seismic design. Before performing the analysis, the modal analysis was carried out in order to obtain the fundamental period of vibration (T_1) and corresponding node displacement of each storey, which will be used to determine the lateral load acting on each storey. Equation 1 [6] was adopted in order to determine the total base shear force, F_b .

$$F_b = S_d(T_1) \cdot m \cdot \lambda \tag{1}$$

where $S_d(T_1)$, m , and λ correspond to the ordinate of the design spectrum at period T_1 , the total mass of the building above the foundation or above the top of a rigid basement, and the correction factor, respectively. The total masses, m of all frames had been calculated as proposed by Rozman and Fajfar [12]. Since the building used in this study only has 2 storeys, the value of λ is equal to 1.

Although Malaysia is located on a stable part of the Eurasian plate, buildings built on soft soil are occasionally subjected to tremors due to far-field earthquakes in Sumatra [13]. Therefore, the Type 1 response spectrum of Eurocode 8 [6] compatible with Soil D had been developed based on all selected PGA as mentioned in previous paragraph. Figure 3 depicts the design response spectrum for behaviour factor, $q = 1.0$ used to determine the base shear force, F_b .

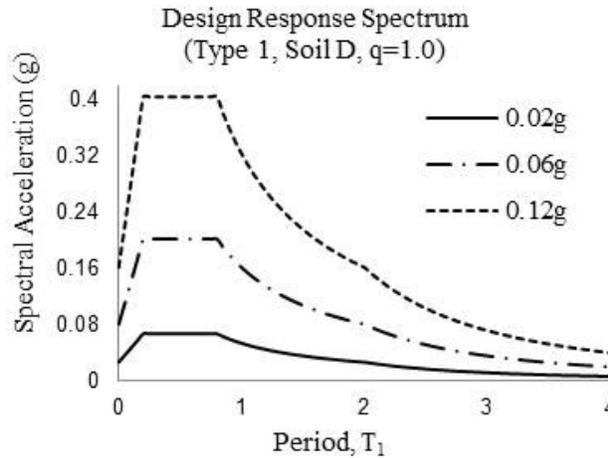


Fig. 3. Design Response Spectrum for Type 1, Soil Class D, $q = 1.0$.

In this work, all frames had been evaluated using nonlinear time history analysis with total 10 ground motion records classified as far-field earthquake as shown in Table 2. Most of it had been used in previous research [5] and are downloadable from strong motion database of the Pacific Earthquake Engineering Research (PEER) Centre [14]. All records can be said as fairly broad as its PGA ranges between 0.0103 g to 0.274 g with magnitude M_w from 6.2 to 7.6. Since only soil type D is considered in this work, all ground motions had been recorded from soft soil with shear wave velocity, $V_s < 180$ m/s.

Table 2. List of Far-Field Ground Motions.

No.	Earthquake	Station name	PGA (g)	Dist. (km)	Mag
1	Chi-Chi, Taiwan	KAU045	0.0103	119.22	6.2
2	Duzce, Turkey	Ambarli	0.038	193.3	7.1
3	Morgan Hill	58375 APEEL 1	0.046	54.1	6.2
4	Morgan Hill	58375 APEEL	0.068	54.1	6.2
5	Loma Prieta	58117 Treasure Island	0.1	82.9	6.9
6	Chi-Chi, Taiwan	TCU040	0.123	21.0	7.6
7	Chi-Chi, Taiwan	TCU040	0.149	21.0	7.6
8	Kocaeli, Turkey	Ambarli	0.184	78.9	7.4
9	Loma Prieta	1002 APEEL 2	0.22	47.9	6.9
10	Loma Prieta	1002 APEEL 2	0.274	47.9	6.9

To simulate the multiple earthquake or repeated earthquake event, the artificial ground motions had been generated. Random combination of single far-field earthquake as listed in Table 2 has been adopted to generate the artificial repeated earthquakes with appropriated scale factor as suggested [4, 9, 15, 16]. Figure 4 depicts the example of time history records of single and repeated earthquakes. Each artificial repeated earthquakes used in this work contain the fore shock, main shock, and after shock component. Very useful RUAUMOKO [17] computer software had been used to conduct the nonlinear time history analysis.

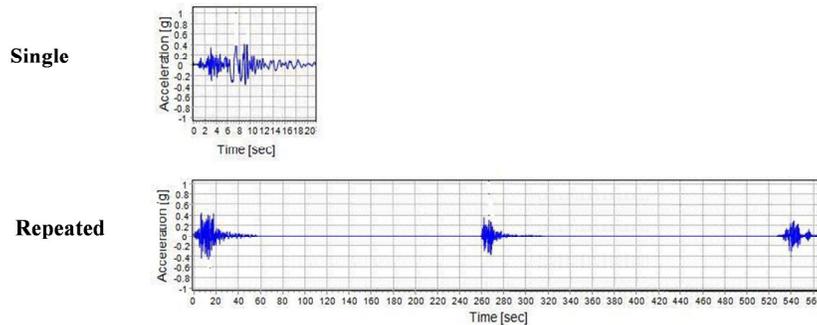


Fig. 4. Typical Profile of Generated Ground Motion.

3. Results and Discussion

3.1. Cost estimation of designed RC frames

In reality, it is hard to establish the additional cost of providing seismic resistance since buildings tend to be unique projects with different layout and requirement [18]. However, it is worth to conduct a study on seismic design and costing so the authority can plan and decide for future development. It is also important in order to give better understanding to designers on optimizing their design so the cost is affordable. In this work, one frame namely as N2BS had been design according to BS8110 [3] to represent current practice of RC design in Malaysia. One frame had been design based on Eurocode 2 [10] without seismic consideration namely as N2EC2. In order to investigate the effect of seismic design to the cost of material, six other frames had been designed based on Eurocode 2 [10] for DCL where seismic load was determined according to Eurocode 8 [6]. It is observed that all frames are differing in term of amount of materials and total cost as discussed in the following section.

3.1.1. Comparison on total volume of concrete used for all frames

The art of designing RC elements, either beam or column is quite unique where the designer have to play around with size of section and the required amount of steel reinforcement, with limitation of minimum and maximum area of total reinforcement as permitted by codes [3, 6, 10]. For example, higher depth of beam section selected by the designer will result in lower amount of steel reinforcement and vice versa. Figure 5 depicts the comparison of total volume of

concrete used in beam and column design for all frames. The size of section for RB, FB and column of N2BS frame is 200 mm × 450 mm, 200 mm × 500 mm, and 250 mm × 250 mm, respectively. It is observed that the normalised total volume of concrete for all beams is equal to one for six frames except frames designed for seismic with reference peak ground acceleration, a_{gR} equal to 0.12 g, which is the highest intensity located in Eastern part of Sabah [2, 11]. This result indicates that the size of beam is remain constant even considering seismic load with reference peak ground acceleration, a_{gR} up to 0.06 g.

However, the size of column start to increase as the reference peak ground acceleration, a_{gR} equal to 0.06 g had been used to determine the seismic load. As for beam, column for both N2q1.0-P2 and N2q1.5-P2 might be designed to have similar size of section with N2EC2 but when the amount of steel reinforcement exceeding the maximum limit at 4% of total concrete area [10], the size of section have to be enlarged. The same limit also applicable for beam designed with DCL. In this work, the total volume of concrete increases around 1.7 and 3.2 times for beam and column, respectively at region with highest seismicity. All frames which their sizes of section have to be enlarged become more rigid with lower fundamental period of vibration, T_1 . Sometimes, the enlargement of section also may create conflict with architectural requirements. From Fig. 5, it is also observed that the level of behaviour factor, q not affecting the total volume of concrete used for design.

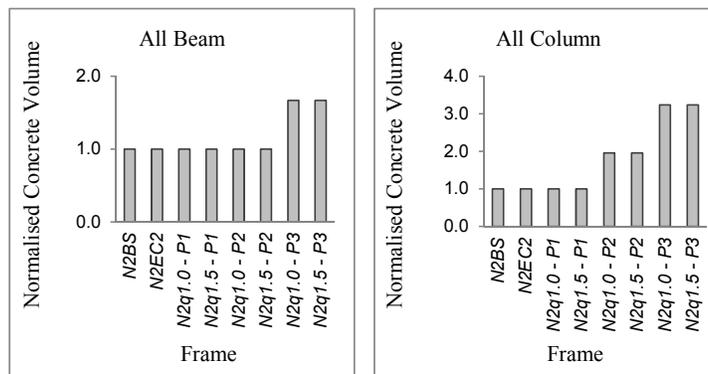


Fig. 5. Comparison of Concrete Volume Used in Design.

3.1.2. Comparison on area of steel reinforcement provided for all elements

Figures 6(a) and 6(b) depict the comparison of total area of steel reinforcement provided, A_s Prov. for beam and column, respectively of all frames normalised to N2BS. In beam, near the exterior and interior support, the tension region located at top part of the section. Due to sagging moment, the tension region move to bottom part of the section at mid span. In this work, the A_s Prov. is just to pass the area of steel required induced by bending moment or to exceed the minimum area of steel [10], whichever is greater.

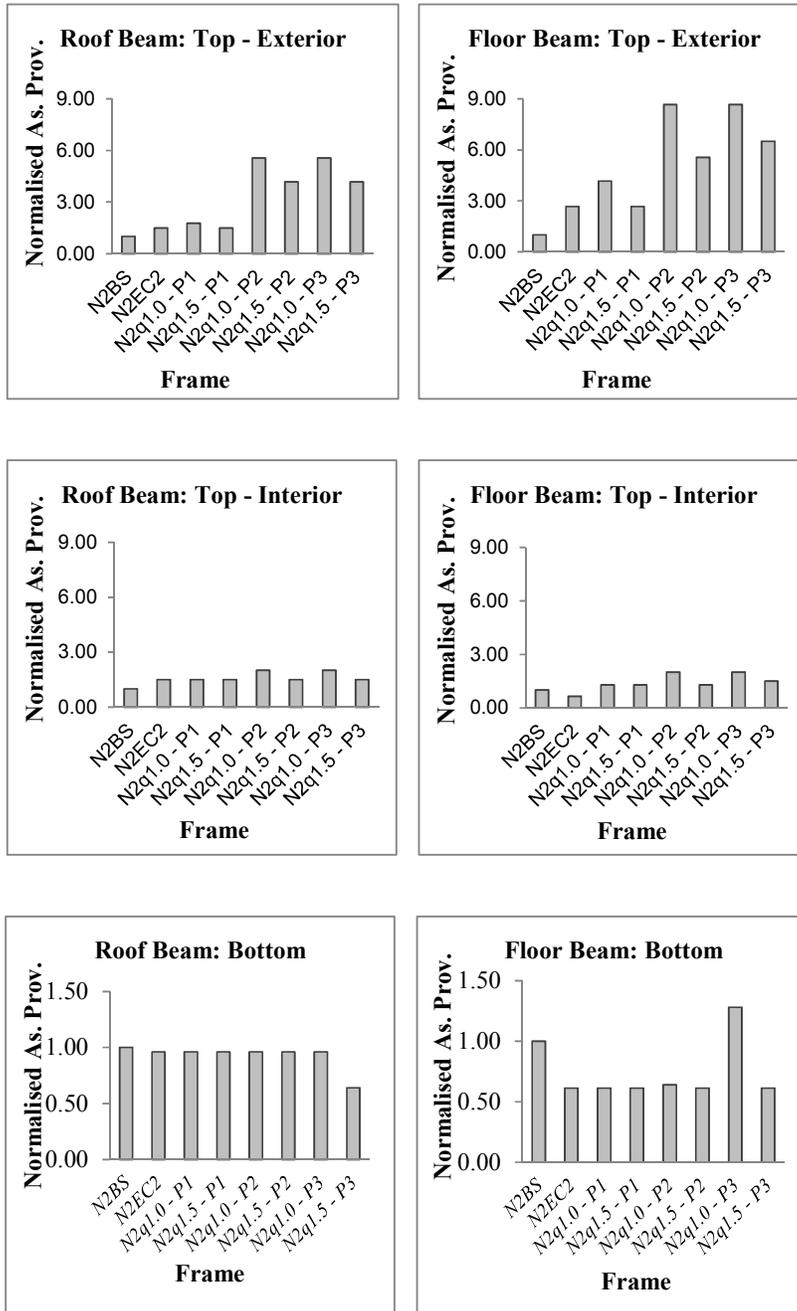


Fig. 6(a). Normalised Area of Steel Reinforcement Provided, As. Prov. for All Beams.

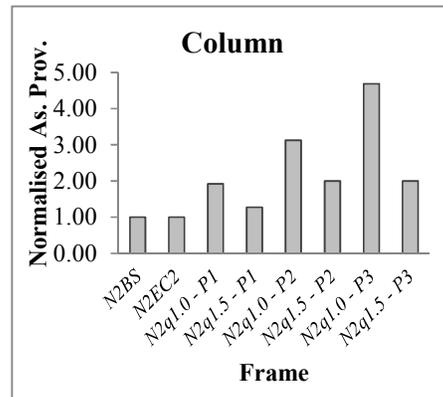


Fig. 6(b). Normalised Area of Steel Reinforcement Provided, As. Prov. for All Columns.

From Fig. 6(a), the As. Prov. as tension reinforcement for beam near exterior support is rapidly increases for both RB and FB especially when seismic load is considered in design. In this region, the As. Prov. is around 5.6 and 8.7 times higher compared to N2BS for RB and FB, respectively. The same trend also observed for As. Prov. for beam near interior support but with lower increment. This result is due to increasing of hogging moment in beam near exterior and interior support when subjected to lateral load. As previously discussed, when the size of section is constant, the number of bar or size of steel reinforcement have to be increased resulting in increment of As. Prov. Due to different size of section, similar amount of As. Prov. near exterior and interior support is observed for all beams of N2q1.0-P2 and N2q1.0-P3 frames. At these regions, higher level of behaviour factor, q considered in design resulting in lower amount of As. Prov.

In Fig. 6(a), it is also observed that total As. Prov. for tension region at mid span of RB is looked constant for all frames except N2q1.5-P3 which have lower As. Prov. associated with larger size of section. With exception for the case of N2q1.0-P3, the As. Prov. at mid span of FB for all frames is relatively around 40% lower compared to N2BS. It can be said that seismic load did not affecting the total amount of As. Prov. at mid span of beam when designed based on low level of reference peak ground acceleration, a_{gR} . For column, the total As. Prov. is rapidly increase when seismic load is considered in design especially frames with behaviour factor, q equal to 1 as shown in Fig. 6(b). Although the size of section had been enlarged as discussed in previous section, the total As. Prov. also have to be increased to resist very high moment in column induced by seismic load. Therefore, it can be concluded that total amount of As. Prov. for column and beam near exterior support is strongly influenced by intensity of seismic load.

3.1.3. Comparison on total weight of steel reinforcement provided for all frames

In RC design, the amount of steel reinforcement to be provided are strongly related to the bending moment and shear force developed from load imposed on frame. The main bar which is also known as longitudinal bar is supplied to resist

tension on concrete section due to action of bending moment. The link or transverse reinforcement is designed to resist shear force. Figure 7 presents the normalised total weight of both reinforcement in beam and column of all frames.

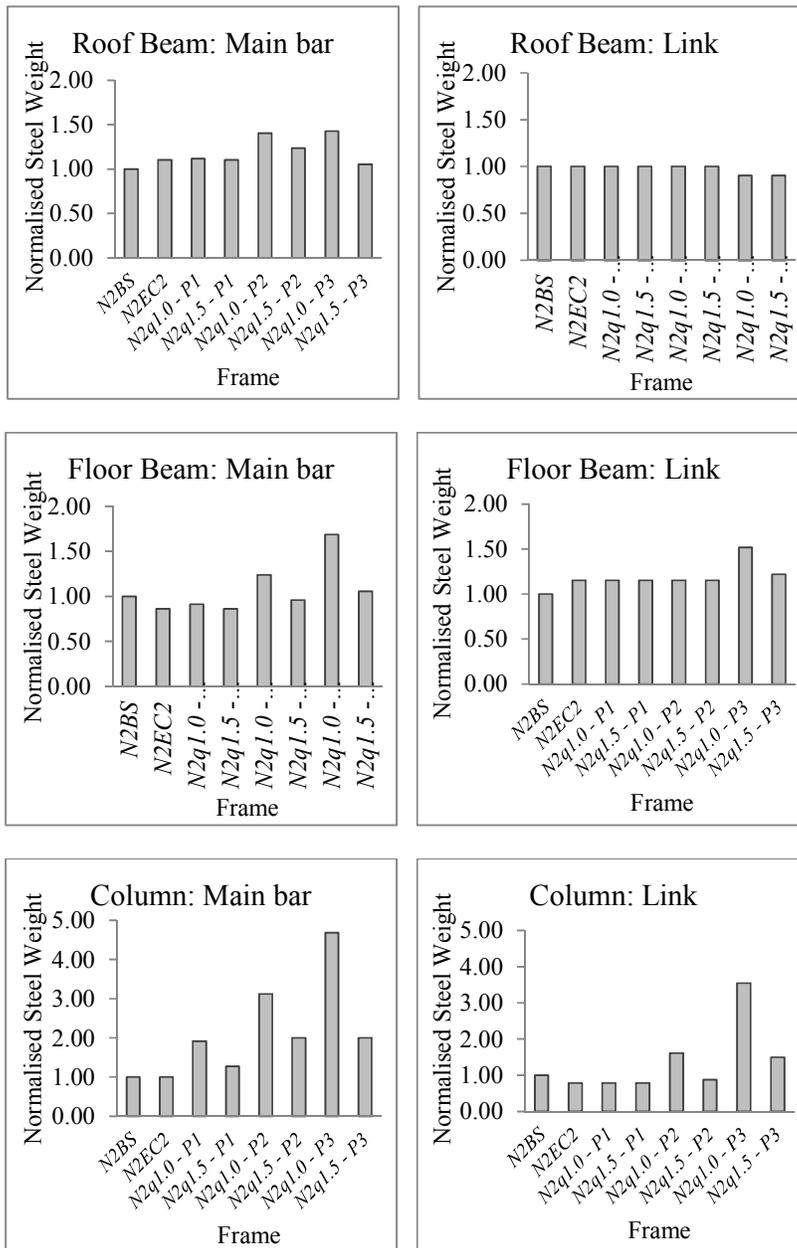


Fig. 7. Normalised Weight of Steel Provided for Each Elements.

It can be observed that total weight of main bar in RB is increase in range of 5 to 40 % higher for frames designed with seismic load. This result is strongly associated with the rapid increment of total As. Prov. at tension region in RB near exterior supports as discussed in previous section. The same trend also observed on main bar of FB especially frames which had been designed at higher level of reference peak ground acceleration, a_{gR} . As the bending moment in column increases rapidly due to action of seismic load, the total weight of main bar provided for frames with seismic design is around 1.9 to 4.7 times higher compared to non-seismic designed frames. It is clearly observed that higher level of behaviour factor, q tends to reduce the total weight of main bar provided for beam and column due to lower magnitude of base shear force, F_b distributed along the height of the frames.

From Fig. 7, total weight of link for RB are almost constant for all frames except N2q1.0-P3 and N2q1.5-P3 which is slightly lower due to larger size of section used in design. For FB, total weights of link for all frames designed based on Eurocode 2 [10], either considering seismic load or not are higher compared to N2BS. This result is in line as explained by Bhatt et al. [19], where Eurocode 2 [10] requires a greater amount of minimum links than BS 8110 [3]. Total weight of link for column is increase as the reference peak ground acceleration, a_{gR} considered for design is increase due to higher shear force to be resisted. According to Eurocode 2 [10], the size of link for column is depend on size of main bar provided, which is equal to $\frac{1}{4}$ of diameter of main bar or 6 mm, whichever is greater. Maximum spacing of link in column also influenced by 20 times diameter of main bar, dimension of column, or maximum limit at 400 mm. Then, the minimum value of the three is selected as maximum spacing.

3.1.4. Cost estimation for all frames

From total volume of concrete and steel reinforcement provided, the total cost of materials can be estimated. Figure 8 depicts the estimated cost of the whole frame normalised to the current practice represented by N2BS. As discussed in previous section, frames designed with seismic design require higher volume of concrete and weight of steel reinforcement compared to non-seismic frames. Therefore, in Fig. 8 it is observed that the total cost for frames with seismic design is around 1.06 to 2.74 times higher compared to N2BS.

The total cost is strongly influenced by the level of reference peak ground acceleration, a_{gR} and behaviour factor, q . Therefore, the same building may have different cost of material for superstructure when built in regions with different level of PGA. As an example, the increment of total cost of material of two storey RC building with seismic design built in West Coast of Peninsular Malaysia (PGA = 0.08) and East Coast of Sabah (PGA = 0.12 g) is 2.1 and 2.7, respectively when the behaviour factor, q is equal to 1. In context of Malaysian economic consideration, very high increment in total cost is unacceptable. Besides, it is not economically feasible to design structures to respond elastically during earthquake [20]. For reason of economy, most structures are designed to behave inelastically under strong earthquake. Hence, lower level of behaviour factor, q up to 1.5 is allowed for design of structure with DCL [10]. When considering behaviour factor, q equal to 1.5, the increment of total cost of the same frame at West Coast of Peninsular Malaysia and East Sabah is around 1.5 and 1.72,

respectively higher than current practice. If such increment is still unaffordable, higher class of ductility may be considered for both regions.

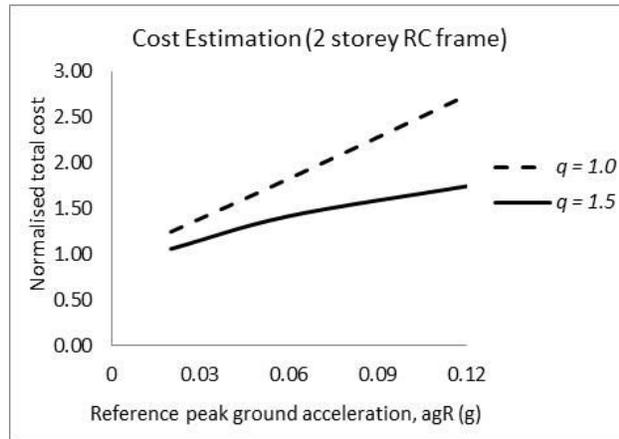


Fig. 8. Estimated Cost Normalized to Current Practice without Seismic Design.

3.2. Seismic performance of designed RC frames

Interstorey drift ratio (IDR) is a good indicator in order to evaluate the structural performance when subjected to earthquake load. The IDR corresponds to the relative lateral displacement between two adjacent storey normalized to its storey height. The accurate estimation of IDR is very important for purpose of seismic performance evaluation since the structural damage is directly related to the magnitude of IDR [7]. Thus, in this paper the mean value of IDR from 10 ground motion records had been used to present the seismic performance of all frames subjected to single and repeated earthquakes.

3.2.1. Seismic performance of RC frames designed without seismic consideration

As mentioned in earlier section, RC design in Malaysia only consider the gravity load for low rise building as used in this work. The suggestion to implement the Eurocode 2 [10] to replace current practice of BS 8110 [3] also creates an option in RC design. Thus, it is good to compare the seismic performance of similar RC frames where both codes [3, 10] had been referred for design without considering seismic action. Figure 9 depicts the IDR of both N2BS and N2EC2 frames when subjected to earthquake with PGA equal to 0.06 g.

Figure 9, it is observed that N2BS frame experienced higher IDR compared to N2EC2 frame when subjected to single earthquake excitation especially at upper level. This result is believed to be associated with higher amount of steel reinforcement near the exterior support in beam for N2EC2 frame which was designed according to Eurocode 2 [10] as discussed in previous section. One more reason is related to the design consideration where the yield strength, f_y of steel reinforcement for shear is higher for the N2EC2 frame compared to the N2BS

frame due to using ribbed bar. Therefore, the N2EC2 frame became stronger compared to another one. When subjected to repeated earthquake excitation, the magnitude of IDR experienced by both frames is relatively higher compared to the single earthquake. As shown in Fig. 5, it can be clearly observed that the repeated earthquake had caused increment in IDR around 19-24% for both frames. The magnitude of IDR for both frames also looked to become almost similar when subjected to repeated earthquake. This result indicates that the repeated earthquake is possible to cause more damage on structures as previously discussed [8].

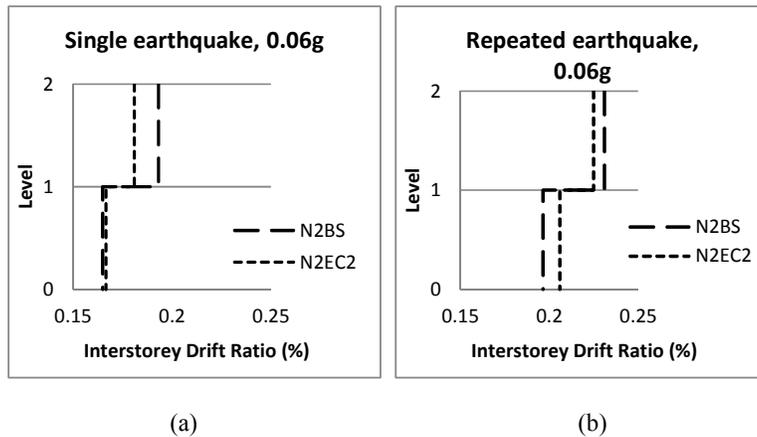


Fig. 9. Interstorey Drift Ratio of Non-seismic RC Building
(a) Single (b) Repeated Earthquakes.

3.2.2. Seismic performance of RC frames designed with different level of PGA

Figure 10 presents the seismic performance of two storey RC frames with seismic consideration in design when subjected to earthquake at intensity of PGA equal to 0.06 g. For comparison, the IDR for both N2BS and N2EC2 frames also presented. To cover the whole area of the nation, 3 different level of PGA had been considered as reference peak ground acceleration, a_{gR} which is equal to 0.02 g, 0.06 g, and 0.12 g as mentioned in previous section. Hence, three different frames had been designed by considering fix behaviour factor, $q = 1$. As discussed before, the design is strongly influenced by the level of PGA used as reference peak ground acceleration, a_{gR} . From Fig. 10, it can be observed that seismic performance, in term of IDR also strongly affected by the same parameter. All frames with seismic design performed better than non-seismic frames with lower IDR reflecting lower lateral displacement induces by earthquake. As expected, frame N2q1.0-P3 which designed based on highest level of reference peak ground acceleration, a_{gR} , experienced the lowest IDR compared to the others. This is due to higher strength designed for the frame to resist high bending moment. The frame also became stiffer as it had been designed with bigger size of section which caused the lateral displacement becomes smaller.

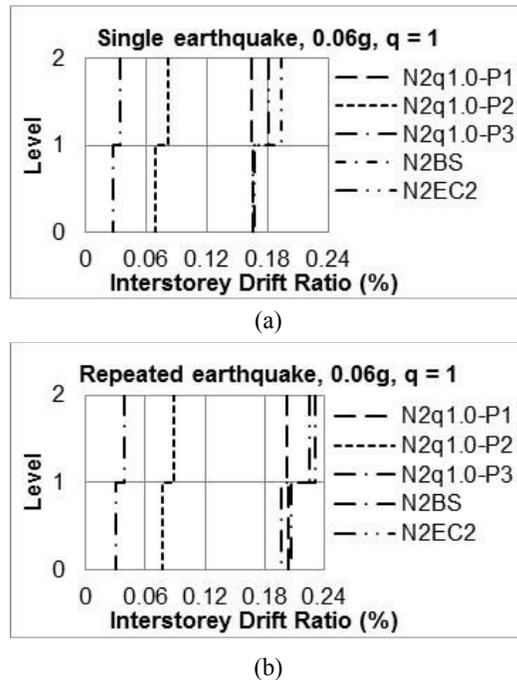


Fig. 10. Interstorey Drift Ratio for Frames with Seismic and Non-seismic Design (a) Single (b) Repeated Earthquakes.

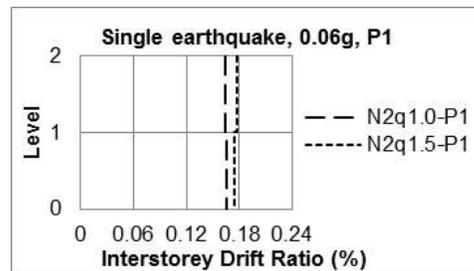
It is also observed that the IDR of all frames with seismic design is evenly distributed along the height compared to non-seismic frames. This result indicates that the relative displacement between two adjacent storey is almost similar at level 1 and level 2 for frames with seismic design, which is not occur to frames without seismic consideration. When $PGA = 0.02$ g had been used as reference peak ground acceleration, a_{gr} , for design the IDR experienced by frame N2q1.0-P1 is almost similar with non-seismic frames of N2BS and N2EC2 at level 1. This result is associated with the design which is almost similar between all 3 frames, especially the size of section. However, due to higher amount of column longitudinal reinforcement provided for frame N2q1.0-P1, the IDR is evenly distributed between level 1 and level 2.

When considering the repeated earthquake excitation, the magnitude of IDR for all frames is relatively higher compared to its corresponding IDR caused by single earthquake. This result is agrees well with previous study by Hatzigeorgiou and Liolios [21] which concludes that the interstorey drift generated by repeated earthquake is larger than that caused by single earthquake. The effect of repeated earthquake on IDR is clearer for weaker frames, especially the non-seismic frames of N2BS and N2EC2 and also frame N2q1.0-P1 which consider the lowest reference peak ground acceleration, a_{gr} , in design. As all aforementioned frames have around 19-24% higher IDR when considering repeated earthquake, both frame N2q1.0-P2 and N2q1.0-P3 only experienced 10-16% increment of IDR. Hence, it can be said that the weaker structure is imposed to greater damage when subjected to repeated earthquake compared to the stronger one.

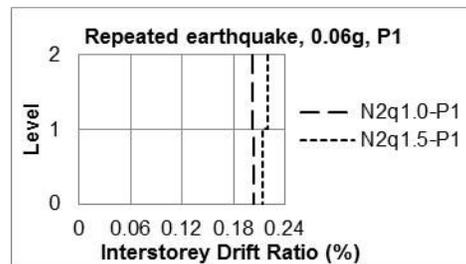
3.2.3. Effect of behaviour factor, q , on seismic performance

As discussed in previous section, increasing behaviour factor, q from 1 to 1.5 produced RC frame with lighter design in steel reinforcement although have the same size of section. Figure 11 depicts the IDR experienced by two RC frames designed with different level of behaviour factor, q when subjected to earthquake with intensity of PGA equal to 0.06 g. As observed, frame N2q1.5-P1 designed based on higher behaviour factor, q have around 8% higher IDR compared to frame N2q1.0-P1 when subjected to single earthquake. This result is as expected because adopting higher level of behaviour factor, q makes the frame becomes more ductile and allow it to sway at higher magnitude. Besides the economical reason, ductile structures is essential in seismic design which available for large absorption and dissipation of energy from earthquake action [22]. From Fig. 11, it is also observed that although using different level of behaviour factor, q the IDR is evenly distributed along the height for both frames. This result is associated with higher amount of steel reinforcement provided at critical region especially near the exterior beam ~ column joint when seismic action is considered in design. Therefore, the strength is evenly distributed to the whole structure and not concentrated at specific storey, i.e. the bottom storey as in case for non-seismic frames.

From Fig. 11, the IDR of both frames caused by repeated earthquake also higher compared to their corresponding single earthquake. Again, frame N2q1.5-P1 which had been design with higher behaviour factor, q has higher IDR. However, the increment of IDR for both frames is similar, which is around 23%. This result indicates that the level of behaviour factor, q did not affecting the increment of IDR caused by repeated earthquake at low intensity of PGA.



(a)



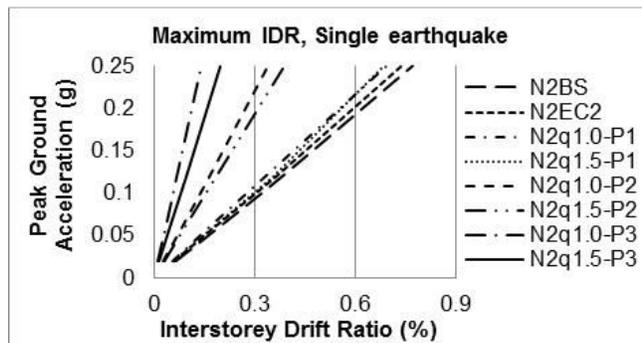
(b)

Fig. 11. Interstorey Drift ratio for Frames with Different Behaviour Factor, q , (a) Single (b) Repeated Earthquakes.

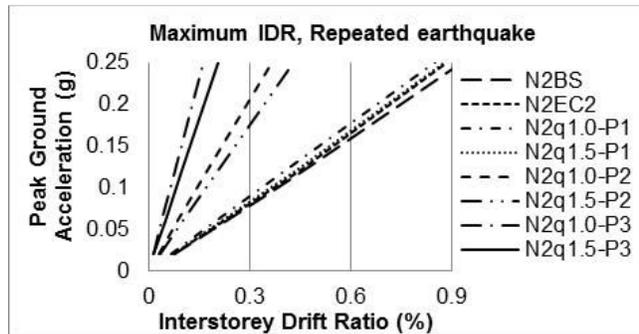
3.2.4. Maximum interstorey drift ratio at different intensity of PGA

In this work, all frames had been evaluated using nonlinear time history analysis at 4 different intensity of PGA namely as 0.02 g, 0.06 g, 0.12 g and 0.25 g to study their capacity against earthquake load, either single or repeated. Figure 12 depicts the maximum IDR for all frames at various intensity of PGA for both single and repeated earthquakes.

It can be clearly observed that the magnitude of IDR increase linearly with intensity of PGA for all frames. As expected, at same intensity of PGA the IDR of non-seismic frames are higher compared to those with seismic design. At PGA lower than 0.1 g, the IDR of N2q1.0-P1 and N2q1.5-P1 frames are looked almost similar with response of non-seismic N2BS and N2EC2 frames due to similar size of section for beam and column. When the intensity of PGA increases, the IDR of both frames with seismic design are lower due to higher amount of steel reinforcement provided. As discussed in previous section, by considering higher level of peak ground acceleration, a_{gR} for design resulting in larger section and higher amount of steel reinforcement. Therefore, the frames become more rigid and the lateral displacement due to earthquake becomes lower. This is important to ensure human comfort [22], but the cost to construct such buildings also has to be considered.



(a)



(b)

Fig. 12. Maximum IDR at Various Level of PGA
 (a) Single (b) Repeated Earthquakes.

If repeated earthquake is subjected, the maximum IDR of the same frame is relatively higher compared to single earthquake. This result is obtained for all frames regardless design consideration taken into account. The effect of repeated earthquake on IDR becomes clearer for weaker frames as the intensity of PGA increases, especially when the PGA greater than 0.05 g. The difference of maximum IDR for same frame designed with different level of behaviour factor, q also increases when the PGA increases.

4. Conclusions

A total 8 number of 2 storey RC buildings for general office use had been designed according to BS8110 [3] and Eurocode 2 [10] with and without seismic consideration to study the increment of cost of material if seismic design has to be implemented in Malaysia. Three different level of reference peak ground acceleration, a_{gR} had been considered in design to represent the whole seismic region in Malaysia as reported [2, 11]. Since this work only considers DCL for seismic design, the level of behaviour factor, q used is 1 and 1.5 as proposed in Eurocode 8 [6]. Then, all frames had been evaluated using nonlinear time history analysis at 4 different intensity of PGA namely as 0.02 g, 0.06 g, 0.12 g and 0.25 g to study their capacity against earthquake load. Both cases of single and repeated far-field earthquakes had been considered in the analyses.

From this work, the following conclusion may be drawn:

- The total volume of concrete is strongly influenced by the level of reference peak ground acceleration, a_{gR} used in design 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.
- The same factor also affecting the increment of total weight of steel reinforcement to be provided for frames with seismic design especially in column and beam near the exterior supports.
- By considering behaviour factor, q equal to 1 in design leads to unacceptable increment in total cost of material, which is up to 270% higher compared to current practice using BS8110 [3]. When higher level of behaviour factor, q equal to 1.5 had been taken into account, it only cause the increasing of cost in range of 6 to 72% depend on seismic region. However, it is important to notice that these results had been obtained by considering response spectrum for Soil Type D according to Eurocode 8 [6] to determine the base shear force, F_b as lateral load. Lower increment of total cost of material is expected for other soil types due to lower proposed soil factor [6].
- From nonlinear time history analyses, it had been proved that the repeated earthquake tends to induce around 8 to 29% higher IDR compared to single earthquake especially on weak frames. However, the IDR looked to be similar between both cases when the intensity of PGA is lower than 0.05 6g g.
- Frames designed based on higher behaviour factor, q tend to experienced higher IDR due to lower strength provided even have the same size of section for all elements.

At the moment of this paper is written, a comprehensive work is conducted to re-design the same frame but considering other class of ductility, which is

ductility class medium (DCM) based on Eurocode 8 [6] with various level of higher behaviour factor, q . Frames with higher number of storey also will be taken into account in future works.

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