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Impact of Seismic Design on Steel Reinforcement and Cost of Structural Works for Medium Rise Apartment

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ABSTRACT

The Sabah region in East Malaysia has been officially classified as a region with active local seismic faults. The region of Ranau and Kundasang saw a significant seismic event on June 5th, 2015, characterised by a magnitude of M_w 6.1. This earthquake resulted in the unfortunate loss of 18 lives and damaged 61 structures. The damages can be attributed to the structural deficiencies that rendered them incapable of withstanding seismic forces. Implementing seismic design is necessary for new buildings to guarantee the general public's safety. Nevertheless, this practice possesses advantages and disadvantages, notably concerning financial implications. The present study aims to investigate the influence of seismic considerations on the expenses associated with steel reinforcement and structural work. This will be achieved using the Malaysian National Annex to Eurocode 8 as a reference. The present study used an 8-story residential building as a representative model. This study examined the two soil types, C and E, in the Sabah region. Various levels of seismicity have been simulated by considering the reference peak ground acceleration, α_{gR} , within the range of 0.04g to 0.16g. The findings indicate that incorporating earthquake considerations into the design process led to an average increase in the utilisation of steel reinforcement by approximately 3% to 110% for beams, columns, and lift cores. Nevertheless, the extent of the rise in expenses related to structural work due to seismic design varied between 1.0% and 18.0%, depending on factors such as soil type and seismic activity levels, compared to the model that did not incorporate seismic considerations. Therefore, using seismic design principles in constructing new buildings merits careful consideration, as it can enhance their resilience and mitigate the financial burden associated with earthquake-related repairs and losses.

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1. Introduction

The belief that Malaysia is exempt from seismic activity is now obsolete. Malaysia is not immune to earthquakes. The nation can be considered a low-seismicity region, except for Sabah, where earthquakes are locally known to occur [1]. Over the previous 140 years, the region of Sabah in East Malaysia has observed a notable escalation in the frequency and intensity of seismic activity, specifically earthquake events [2]. The 2004 Aceh earthquake also affected Peninsular Malaysia. High seismic waves from the event caused extensive shaking of high-rise buildings, especially in Johor Bahru, Kuala Lumpur, Penang, and Putrajaya [3]. In addition, the 2004 Aceh earthquake also triggered the reactivation of the Paleo fault line [4]. As a result, a small series of earthquakes occurred in Bukit Tinggi and Janda Baik from 2007 to 2009. The localities of Jerantut and Manjung in Peninsular Malaysia also encountered a seismic event.

In the context of Malaysia, it is noteworthy that the region of Sabah has a historical record of seismic activity. Kundasang, Lahad Datu, Pitas, Ranau, and Tawau regions have been identified as areas with a heightened susceptibility to seismic activity. In 1976, a seismic event of a magnitude of M_w 5.8 took place in Lahad Datu. Almost 40 years later, on June 5th, 2015, a M_w 6.1 medium earthquake hit Ranau, originating about 16 km northwest of Ranau City. There were more than 100 aftershocks [5] following the earthquake. The main shock caused 18 deaths and damage to many buildings. According to the initial assessment by [6], various structural elements of reinforced concrete (RC) buildings, including beams, columns, and beam-column junctions, sustained damage. These affected structures encompassed residential quarters, educational institutions, and places of worship, including mosques and temples. The study also revealed that the Weak Columns, - Strong Beams and Soft Storey are the main reasons for structural damage. This damage mechanism was strongly associated with previous construction practices in Malaysia, which did not consider seismic action during the design stage [7-9]. The damages during the 2015 Ranau earthquake had proven earlier findings, which concluded that most RC columns for Malaysian buildings were inadequate to resist earthquake loads [1]. As a result, local authorities in Malaysia must adopt the seismic design for new buildings [10-11] to prevent severe damage during future earthquakes.

By the end of 2017, the Malaysian National Annex to Eurocode 8 for seismic design regulations was accessible [12]. However, seismic design practices for new buildings in Malaysia still need to be fully implemented. One of the factors is considering the cost [9]. Several previous studies [8-9, 13-18] have reached the consensus that the incorporation of seismic design considerations in reinforced concrete buildings leads to an escalation in the cost of structural work, primarily attributed to the increased utilisation of steel. However, most past studies considered the soil factor, S based on Eurocode 8 [19] instead of referring to the proposed value in the Malaysia National Annex [12]. Except for the studies conducted by [8, 13, 15, 20-21], previous research works have predominantly employed the Lateral Force Method for seismic analysis, primarily due to the structural models' regularity. For the same reason, past studies considered the behaviour factor, $q = 3.9$, for Ductility Class Medium (DCM) design consideration. This study used an 8-story RC residential building, which was irregular in plan and elevation, as a model. Therefore, the modal response spectrum analysis has been adopted in seismic analysis for design purposes with an adjusted value of behaviour factor, q . This study also refers to the latest schedule of rates, which the Malaysian Department of Work officially released for costing purposes [22].

2. Models and Methods

Three distinct stages were executed to fulfil the aims of this study. The initial step involved generating the basic model through structural analysis. Then, the seismic design analysis was generated depending on the soil type and level of seismicity. Lastly, the taking-off method was executed. In stage 1, the basic model of an 8-story RC residential building was generated, as shown in Figures 1 and 2. From the top of the rigid foundation, the overall height, H , was equal to 28.20 m. According to the results obtained from modal analysis, the fundamental period of vibration, $T_1 = 0.918$ s for the first mode.

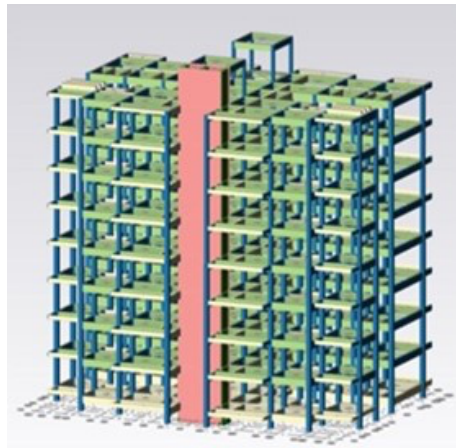


Fig. 1. Whole building view for an 8-story of RC residential building

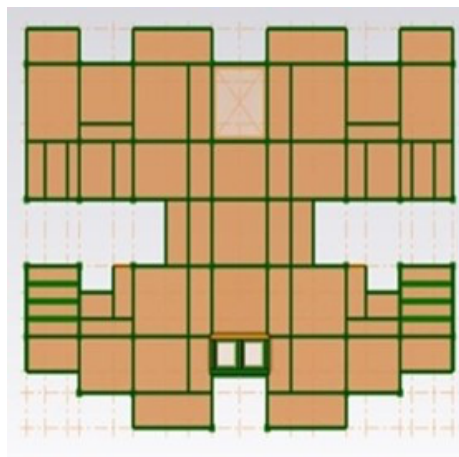


Fig. 2. Plan view of the building for an 8-story RC residential building

2.1 Designing Model Stage

Several beam sizes have been proposed, depending on the position and span. The width of the beams was in the range of 250 mm to 350 mm, while the depth of the beams was in the range of 300 mm to 700 mm. Four different sizes of columns had been proposed for the basic model. The smallest one was (300×300) mm², which was located at the top 5-story levels of the model. The largest column was (450×450) mm², located at the 5-story levels from the bottom of the model. Since the

building was an 8-story building, it was equipped with lift cores with thicknesses of 300 mm to 425 mm, depending on its position.

Upon completing stage 1, the basic model underwent analysis and design following Eurocode 2 [23], without considering seismic design considerations, to represent current construction practices. Wind load had been considered for this model since it was a medium-rise building. The basic wind speed of 32.5 m/s was considered to derive the wind pressure on the building. In addition, the altitude and distance from sea to site were assumed to be 20 m and 10 km, respectively.

The analysis and design were conducted by using Tekla Structural Design computer software. First of all, the modelling process started by adjusting the code of practice to match the Malaysia National Annex [12], Eurocode 2 [23], and Eurocode 8 [19]. The gridline had been set according to the structural layout provided earlier. Then, the beams, columns, and lift walls were assigned one by one. Next, the modelling process focused on the assignment of gravity and wind loads. Finally, the modelling ends with a validation process before proceeding to analysis and design. This model was used as a control model, and the data was used for comparison with models with seismic design.

2.1 Designing Seismic Model Provision Stage

In the second stage of the study, a comparable model underwent a redesign to incorporate seismic protections outlined in the Malaysia National Annex [12] and Eurocode 8 [9]. The variable load, denoted as Q_k , exerted on the slabs exhibits a range of 2 kN/m² to 4 kN/m², contingent upon the specific utilisation of the floor, as stipulated by Eurocode 1 [26]. The primary objective of this study was to examine the impact of soil type and seismicity levels on the utilisation of steel rebar in models designed for seismic conditions. This study examined various soil types and seismicity levels as independent variables in models of seismic design, as previously explored in existing research [8, 14, 16-18, 20-21, 26]. Soil Types C and E in Sabah had been considered alongside five different levels of seismicity. The latter was determined depending on the reference peak ground acceleration value, α_{gR} , referred to as the National Annex [12], as shown in Figure 3. The map shows areas with low seismicity levels: Kota Kinabalu, Kota Marudu, Penampang, and Sandakan. Meanwhile, at the medium seismicity level, the area includes Beluran, Lahad Datu, Merotai, and Ranau.

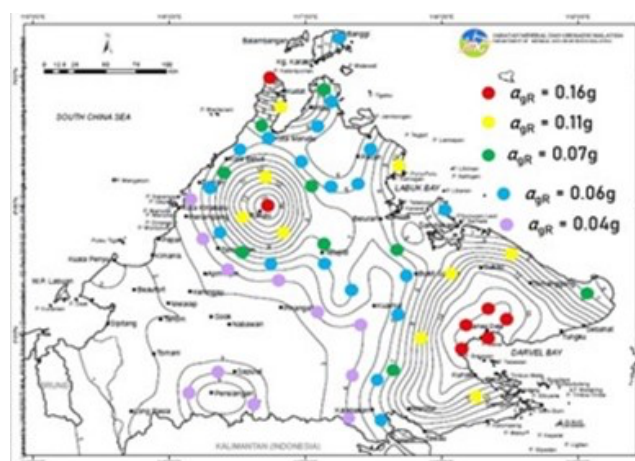


Fig. 3. Seismic hazard map for Sabah [13]

In the case of low seismic activity, the reference peak ground acceleration, α_{gR} , was determined to be between 0.04g and 0.06g, categorised as the Ductility Class Low (DCL). For medium seismicity levels, the reference peak ground acceleration, α_{gR} , equals 0.07g, 0.11g, and 0.16g. The models incorporating reference peak ground acceleration values, α_{gR} , were carried out using the DCM

methodology. Table 1 summarises all the models used in this study and the design considerations. Model number 1, labelled with NS, was the control model, designed with non-seismic provisions. Model numbers 2 to 11 had been designed based on seismic provisions with different values of variable parameters.

Based on the Malaysia National Annex [12] and Eurocode 8 [19], the residential building was categorised as important class III, where the value of the importance factor, γ_1 , equals 1.2. According to [25, 26], an increased value of the importance factor, γ_1 , can provide excellent life protection in disaster situations. As indicated in the previous section, prior research has examined the significance of the behaviour factor, $q = 3.9$, in the context of DCM, owing to the consistent nature of the structural system. In this study, the model used was irregular in plan and elevation. Hence, following Eurocode 8 [19], the behaviour factor, q , had been determined to be 1.5 and 2.76 for DCL and DCM, respectively. In addition, this study used modal response spectrum analysis for design purposes. This method enables a higher fundamental period of vibration, T_1 . The effective stiffness of beams was taken as 0.5 times their elastic stiffness to be considered for cracking. For columns and lift walls, the effective stiffness was taken as 0.7 times their elastic stiffness. Based on modal analysis, the fundamental period of vibration, T_1 , for mode shape 1, was equal to 1.164 sec. The latter was higher compared to the fundamental period of vibration, T_1 , obtained based on Clause 4.3.3.2.2 in Eurocode 8 [19], which was equal to 0.918 sec.

The analysis and design process considers up to 10 mode shapes and a damping ratio of 5%. Each model was designed using steel yield strength, f_{yk} and concrete compressive strength, f_{ck} of 500 N/mm² and 30 N/mm², respectively, by using Tekla Structural Designer computer software. In this study, as listed in Table 1, all models were designed to have similar sizes of corresponding structural elements. This approach has also been used in previous studies [8, 16-18, 20-21] in order to get precise results on the impact of seismic design on the utilization of steel bars. For example, columns at the first 5-story levels had been designed for similar sizes for all models, regardless of the variable parameters. Additionally, in accordance with Eurocode 8 [19], the design limited the inter-storey drift to 0.5% for buildings with non-structural elements made of brittle materials attached to the structure.

Table 1
 Modelling and Design Provisions

No	Model	Soil Type	α_{gR} (g)	Ductility	Behaviour factor, q
1.	NS	-	-	-	-
2.	004-C	C	0.04	DCL	1.5
3.	006-C	C	0.06	DCL	1.5
4.	007-C	C	0.07	DCM	2.76
5.	011-C	C	0.11	DCM	2.76
6.	016-C	C	0.16	DCM	2.76
7.	004-E	E	0.04	DCL	1.5
8.	006-E	E	0.06	DCL	1.5
9.	007-E	E	0.07	DCM	2.76
10.	011-E	E	0.11	DCM	2.76
11.	016-E	E	0.16	DCM	2.76

2.2 Take-off Stage

The last stage was the take-off, determining all materials required to construct the structural elements, namely beams, columns, and lift cores. At this stage, the materials involved were concrete, formwork, and steel. The concrete was measured in the form of total volume, while the formwork

was measured in the form of total area. Lastly, the steel had been measured as a total weight, or tonnage. Results obtained for all models with seismic designs were compared with those of the non-seismic model. The comparison had also been made depending on the summation cost of structural work. In this part, the cost of structural work involving the cost of concrete, formwork, and steel was measured based on the latest Standard of Rate issued by the Malaysian Department of Work [22].

3.0 Results and Discussions

3.1 Base Shear Force, F_b

As previously indicated, the control model was designated NS. The model solely takes into account the effects of gravitational force and wind. In addition, it was worth noting that the models shown in Table II were carefully developed, considering both gravitational and seismic forces. The latter had been assigned lateral action, which acted on each story joint. According to the information provided in Table 2, the cumulative seismic activity was determined as the base shear force, F_b , following the guidelines outlined in Eurocode 8 [19]. Subsequently, the base shear force, F_b , was allocated among the individual story joints, referred to as the lateral story force, F_i .

The equation for the base shear force, F_b , was generated as a mathematical expression, including the spectral acceleration at the fundamental period of vibration, $S_d(T_1)$, the effective mass of the building, m , and the correction factor, λ . The correction factor, λ , was determined to be 0.85 based on the number of stories being greater than two and the fundamental period of vibration being less than twice the characteristic period, $T_1 < 2T_c$ [19]. As previously stated, the dimensions of the appropriate structural components exhibited uniformity across all models. The permanent action, G_k , and the variable action, Q_k , exhibited similarity across all models. Hence, it can be shown that the effective mass of the building, m , had a consistent value across all models, around 6858.5 tonnes.

The influence of the spectral acceleration during the fundamental period of vibration, $S_d(T_1)$, on the base shear force, F_b , was observed in Table II, where it was noted that the effective mass of the building, m , and the correction factor, λ , were consistent across all models. By consulting Clause 3.2.2.5(1) of Eurocode 8 [19], it was possible to determine the spectral acceleration at the fundamental period of vibration, $S_d(T_1)$. The former was subject to considerable influences from multiple elements, such as the fundamental period of vibration, T_1 ; the soil type; the behaviour factor, q ; and the design ground acceleration, α_g . Furthermore, the reference peak ground acceleration value, α_{gR} , and the importance factor, γ_1 , directly impacted the latter.

Table 2
 Seismic Action on All Models

No	Model	Reference peak ground acceleration, α_{gR} (g)	Spectral peak acceleration at the fundamental period of vibration, $S_d(T_1)$ (g)	Base shear force, F_b (kN) Lateral Force Method	Base shear force, F_b (kN) Response Spectrum Analysis
1.	NS	-	-	-	-
2.	004-C	0.04	0.546	3182.3	3064.8
3.	006-C	0.06	0.819	4773.5	4597.2
4.	007-C	0.07	0.519	3026.7	2914.9
5.	011-C	0.11	0.816	4756.2	4580.6
6.	016-C	0.16	1.186	6914.4	6662.4
7.	004-E	0.04	0.472	3235.5	2711.5
8.	006-E	0.06	0.708	4853.2	4067.3
9.	007-E	0.07	0.449	3077.2	2578.9
10.	011-E	0.11	0.705	4835.7	4052.6
11.	016-E	0.16	1.026	7033.7	5894.7

Table 2 shows that the base shear force, F_b , exhibits an upward trend as the spectral acceleration at the fundamental period of vibration, $S_d(T_1)$, increases. This relationship holds regardless of the soil type. This finding was in line with earlier studies by [14, 16-18, 20-21]. The relationship between the spectral acceleration during the fundamental period of vibration, $S_d(T_1)$, and the reference peak ground acceleration, α_{gR} , was evident in that $S_d(T_1)$ increases with higher α_{gR} values. Based on response spectrum analysis, the highest base shear force, F_b , that Model 016-C encountered on Soil Type C was $F_b = 6662.4$ kN. In addition, it was worth noting that model 016-E had the highest base shear force, $F_b = 5894.7$ kN, when investigated on Soil Type E. The findings of this study indicate that structures constructed in places with a higher seismicity level, α_{gR} , were more susceptible to increased lateral forces resulting from earthquakes [14, 16-18].

Hence, the findings were contradictory for models 006-C and 007-C, as well as models 006-E and 007-E. Model 006C exhibited a higher magnitude of base shear force, F_b , although it was associated with lower seismic activity levels than model 007-C. Similar results were also obtained for models 006-E and 007-E. According to previous studies [17, 20], this result was strongly related to various ductility class considerations. Models 006-C and 006-E were designed for DCL, while models 007-C and 007-E were designed for DCM. As previously stated, the behaviour factor values, q , for DCL and DCM were determined to be 1.5 and 2.76, respectively. According to Clause 3.2.2.5 (1) in Eurocode 8 [19], the behaviour factor, q , generally leads to a decrease in the spectral acceleration value at the fundamental period of vibration, $S_d(T_1)$. Therefore, the base shear force, F_b , decreases. It means that models 007-C and 007-E have been subjected to lower lateral action compared to models 006-C and 006-E. This was to ensure models 007-C and 007-E yield faster to consider the non-linear response of higher ductility [19].

According to previous studies [8, 14, 16-18, 20-21, 26], the result in Table 2 also demonstrates that soil type strongly influences the base shear force, F_b . At a similar level of seismicity, α_{gR} models with Soil Type C were continuously subjected to a higher base shear force, F_b . For example, even though it has a similar reference peak ground acceleration ($\alpha_{gR} = 0.16g$), the base shear force, F_b , subjected to model 016-C was 13% higher than model 016-E. The observed outcome exhibited a significant correlation with the varying values of the soil factor, S . Based on the findings of the Malaysia National Annex [12], it was determined that the soil factor, S , for Soil Types C and E in the region of Sabah were recorded as 1.35 and 1.4, respectively. The soil factor, S , for Soil Type C was lower than Soil Type E. Previous research [17] found that at the same level of seismicity, structures exhibiting a reduced fundamental period of vibration, T_1 , and a lower number of stories were usually hit with a higher base shear force, F_b . In this research, however, the models' higher fundamental period of vibration, T_1 , changed the trend. Soil Type C models were constantly subjected to a higher base shear force, F_b , than Soil Type E models. This was due to the characteristic of Soil Type C, which had a higher upper limit of the period of the constant spectral acceleration branch, T_c , compared to Soil Type E in Sabah [12]. This situation results in a higher value of spectral acceleration at the fundamental period of vibration, $S_d(T_1)$, for Soil Type C models. This result strongly indicates that buildings with similar levels of seismicity tend to have different designs if built on different soil types.

3.2 Total weight of steel reinforcement

As mentioned in the earlier section, the amount of steel bars in every structural element was presented as total weight or tonnage. This method of presentation was also used in previous studies [8-9, 13-18, 20-21]. In order to investigate the increase in steel tonnage resulting from seismic design considerations, the total steel tonnage achieved for models incorporating seismic design was

normalised to the total steel tonnage for the non-seismic model. The comparison had been made for beams, columns, and lift cores.

The normalised steel tonnage for beams is presented in Figure 4. The latter shows that seismic design models typically exhibit greater values of steel tonnage compared to non-seismic models. The results were applied to both soil types. This was due to additional action from the forces exerted by the earthquake, which subsequently affected the ground acceleration, α_{gR} , exhibit greater values of steel tonnage. For example, the non-seismic model's beams for model 016-C used around twice the amount of steel. A similar model, considering lower seismicity ($\alpha_{gR} = 0.11g$), used half of the steel of 016-C. A similar pattern was also obtained for models considering Soil Type E. As stated in the previous subsection, models for higher levels of seismicity had experienced a higher earthquake action via base shear force, F_b . The latter exhibits greater values of the bending moment, M , and shear force, V , which results in greater values of steel reinforcement. The outcome obtained exhibited a resemblance to findings reported in other research [9, 17-18, 20-21, 26] but with different increment percentages due to different models and other values of variable parameters.

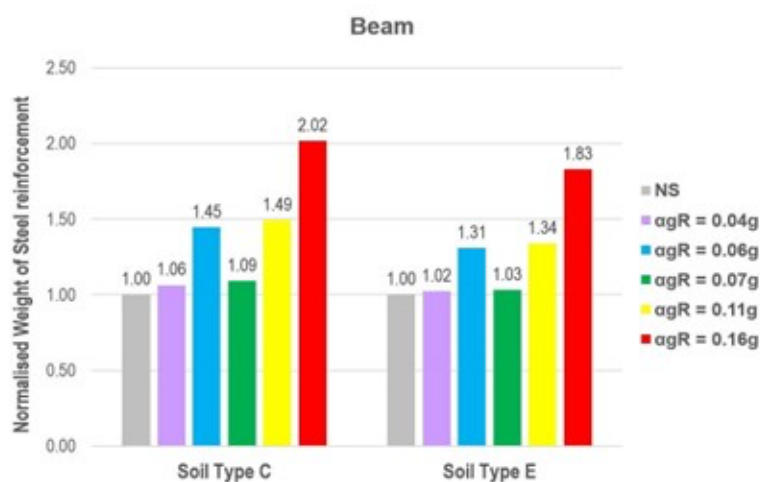


Fig. 4. Normalised steel tonnage for beams

Columns design must be given special attention since it determines the stability of the whole structural system. Columns will carry the vertical gravitational action from beams and transfer it to the foundation. During the earthquake, the columns must resist cyclic lateral action [9]. Therefore, Eurocode 8 emphasises the adoption of the Strong Columns – Weak Beams philosophy in seismic design. Based on this philosophy, the columns must be designed to be at least 1.3 times stronger than the beams connected to them. This approach is essential to avoiding global damage [19].

Figure 5 illustrates the effect of seismic design on the quantity of steel required. The result shows that columns for seismic design models used more steel than the non-seismic model, irrespective of the soil type. However, the increment was insignificant, around 7% for seismic models up to the reference peak ground acceleration, $\alpha_{gR} = 0.11g$. The steel tonnage increased significantly for seismic models with higher seismicity ($\alpha_{gR} = 0.16g$), especially those with Soil Type C. This result was expected because model 016-C was exposed to the greatest base shear force, F_b . Previous studies also observed a similar pattern [16-18, 20-21]. This pattern is related to the philosophy of Strong Columns – Weak Beams. As discussed before, model 016-C had the highest steel tonnage for beams. Therefore, the steel used in its columns also had the highest values to ensure its columns were stronger than its beams. Eurocode 8, with Clause 5.4.3.2.2 (11)b, stated that the minimum area of steel as the columns' main reinforcement should equal 1% of the columns' cross-section [19].

Therefore, except for models 016-C and 016-E, the steel tonnage was almost similar, which was to fulfil the requirement in the clause.

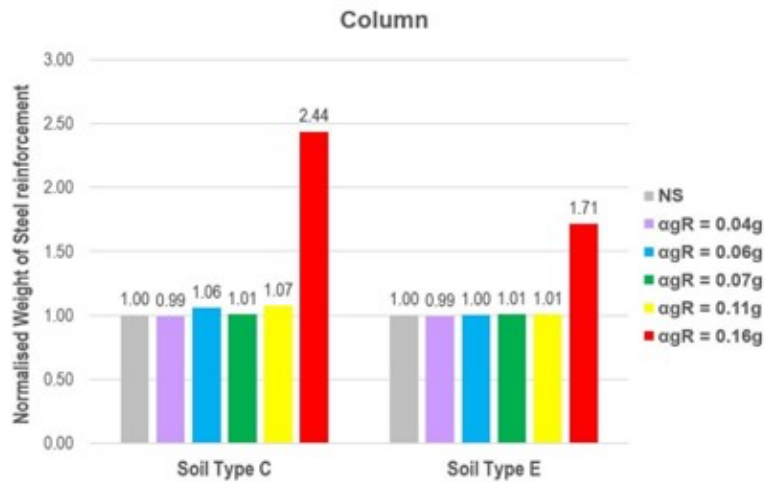


Fig. 5. Normalised steel tonnage for columns

Since it had been classified as a medium-rise building, the models used in this study were also equipped with lift cores. Therefore, the lift cores were explicitly designed as shear walls to resist the lateral action of the earthquake. This meant that the lift cores also functioned as a system of seismic force-resisting. The comparison of steel tonnage for the lift cores is illustrated in Figure 6. Like the pattern for beams and columns, the steel tonnage required for lift cores exhibits an increase in proportion to seismic activity, irrespective of the soil type. The findings of this study resemble the outcomes of the prior investigation [9, 21].

Based on Figure 6, steel reinforcement follows the pattern of the base shear force, F_b . This means that more steel was required for models with a higher base shear force, F_b , to provide adequate strength to lift cores (shear walls) to resist the lateral action. Therefore, the use of steel reinforcement will be higher for construction in regions with higher seismicity. Figure 6 also depicts that the steel tonnages of lift cores for models considering Soil Type C were consistently higher than their corresponding models for Soil Type E. This pattern was tallied with the early discussion on soil type and base shear force, F_b .

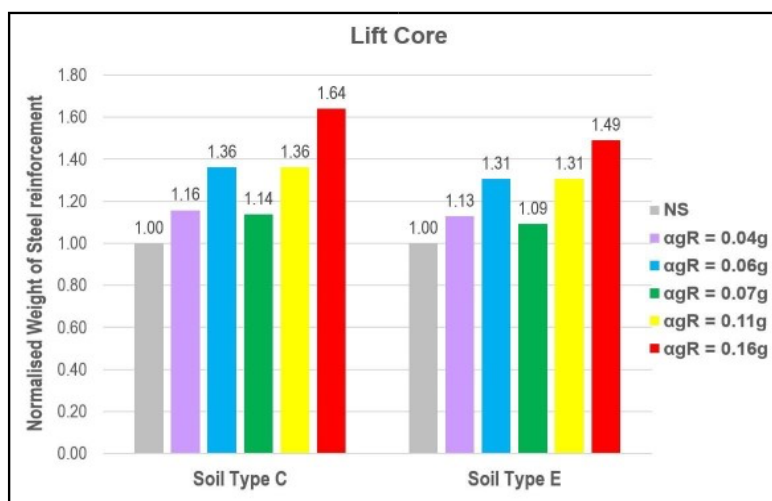


Fig. 6. Normalised steel tonnage for lift cores

3.3 Cost estimation of structural works

This subsection discusses the comparison of the cost of structural works. As mentioned earlier, structural work is the cost of constructing the structural elements, namely beams, columns, and lift cores. The cost of slabs was also considered, even though the design was similar for all models. This represented a complete structural system before the mechanical and architectural finishing work occurred. The measurement of the structural works' costs of concrete, formwork, and steel reinforcement was based on the Standard of Rate by the Malaysian Department of Works in 2021 [22]. Figure 7 presents the cost of structural work for the 8-story apartment models used in this study.

Since the sizes of corresponding structural elements were similar across all models, the summation of the concrete volume and formwork area were also similar for all models. This results in similar concrete and formwork costs for all models. Consequently, the steel reinforcement cost determined the overall structural construction cost. Figure 7 demonstrates that by analysing Soil Type C, the expenses associated with structural works vary between 1% and 18%, based on the level of seismicity. On the other hand, seismic design models increased the cost of structural work from 1% to 13% compared to the model without seismic provision. In this study, model 016-C had the highest structural work cost, around 18% higher than the non-seismic model. This was because the model utilised the most steel due to its high shear force at the base, F_b , as discussed in the previous subsection. Therefore, selecting a suitable site based on geological conditions is also important for economic reasons [9]. Considering the seismic design, the increase in the structural cost was unavoidable. However, a maximum 18% increment for such a building is still worth it because it can save more money for repairing work or loss of operation due to damage or collapse during future earthquakes. The data presented in Figure 7 aligned with the previous study [16-18, 20-21] that suggested a higher peak ground acceleration, α_{gR} , and a higher increment for the structural work cost.

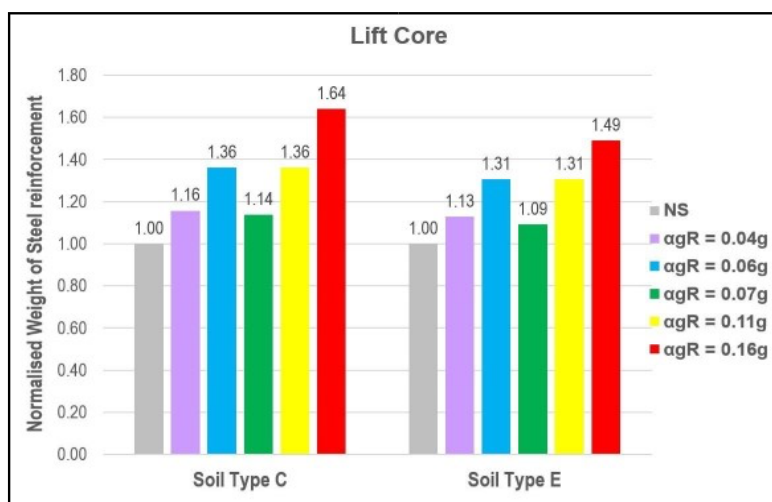


Fig. 7. Normalised cost of structural works

4.0 Conclusions

This research investigated the effect of seismic design on steel rebar. An 8-story apartment in Sabah was used as a model, considering various soil types and levels of seismicity. The National Annex [12] and Eurocode 8 [19] have been referred to for design purposes. For comparison purposes, a non-

seismic control model was designed to be compared with standard practice. The subsequent conclusion has been reached:

- Total steel tonnage for reinforcement tends to increase for seismic design models. The increment depends on the earthquake's lateral action level, as measured by the base shear force, F_b . The latter rises in proportion to the level of seismicity. However, the ductility class also influences the behaviour factor value, q , and impacts the value of the base shear force, F_b . According to the findings, seismic design increases the use of steel rebar by approximately 3% to 110% for beams, columns, and lift cores.
- At a similar level of seismicity, a single structure may have varied applications for steel rebar due to different soil types. In this study, models on soil type C always had higher steel tonnage than those on soil type E. Therefore, the geological aspect and soil profile also must be considered in new developments for economic reasons.
- From a cost perspective, incorporating seismic design measures results in a cost escalation ranging from 1% to 18% for structural works compared to current practices without seismic design. The increment depended on the proposed site's seismicity level and soil type. Such increment might be high; however, it is crucial to consider seismic design principles while constructing new buildings since seismic design will increase the vulnerability of the buildings and reduce the cost of repairing or loss due to earthquakes

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