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Seismic Vulnerability of Educational Facilities in Seismically Active Zones: Case Study of Bengkulu, Indonesia

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ABSTRACT

The province of Bengkulu lies in a highly active seismic zone, so this study aims to evaluate the seismic vulnerability of the Faculty of Economics and Business building at Bengkulu University using an integrated approach that combines site-specific soil response analysis and dynamic structural analysis. Nonlinear soil response analysis was performed to determine surface ground-motion characteristics, while finite-element–based time-history dynamic analysis was used to assess structural behavior. Ground motion records from the 2007 Bengkulu–Mentawai, 1995 Kobe, and 1979 Coyote earthquakes were adopted as seismic inputs. The novelty of this research lies in the application of an integrated approach between site-specific soil response analysis and dynamic structural analysis in evaluating the seismic vulnerability of existing educational buildings, thereby producing a more realistic seismic performance assessment in earthquake-prone areas. The proposed integrated approach offers a more realistic and comprehensive evaluation of seismic performance for educational facilities in seismically active regions. The results show that clay soil layers generate the highest amplification, with the Kobe earthquake producing the maximum peak ground acceleration at the ground surface.

Keywords: *seismic performance; ground response; structural dynamics; Peak Ground Acceleration; earthquake engineering.*

1. Introduction

The province of Bengkulu is located along an active subduction zone on the west coast of Sumatra, resulting in high seismic activity and a high risk of strong earthquakes. Bengkulu Province belongs to the category of Indonesian earthquake-prone provinces, which comprises two strike-slip faults and a subduction zone known as the Mentawai Fault, the Sumatra Fault, and the Sumatra Subduction Zone [1]. The presence of earthquakes is a major risk in the design of multi-storey structures in Indonesia [2]. The Bengkulu-Mentawai earthquake remains the strongest earthquake to have hit Bengkulu City in the past 2 decades. It took place on the 12 th of September 2007 with an 8.6 Mw [3]. In addition to the 2007 Bengkulu-Mentawai earthquake (Mw 8.6), this study used two other major earthquakes as reference loads: the 1995 Kobe earthquake (Mw 6.9) in Japan and the 1979 Coyote earthquake (Mw 5.7) in California, United States. The Mentawai earthquake record represents a large-magnitude subduction event that reflects the characteristics of major seismic sources commonly occurring in Indonesia. The Kobe earthquake produced high peak ground acceleration and significant structural impacts. The Coyote earthquake record is a shallow-crustal earthquake of moderate magnitude with relatively high-frequency content. This study selected these three earthquake ground motions because international studies frequently use them, they provide complete acceleration time-history records, and they have caused significant damage to reinforced concrete structures.

According to[3], the city of Bengkulu is situated near various earthquake sources, as illustrated in Figure 1, where the Sumatra Subduction Zone marks the boundary between the

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Eurasian Plate and the Indo-Australian Plate. Notably, this subduction zone has caused several large megathrust earthquakes along the west coast of Sumatra Island. For instance, the 2007 Bengkulu-Mentawai earthquake struck the Bengkulu region. Furthermore, while the quality of buildings in Bengkulu City is relatively uniform, this suggests that local geological factors, rather than building variability, play a significant role in determining the extent of earthquake damage [4].

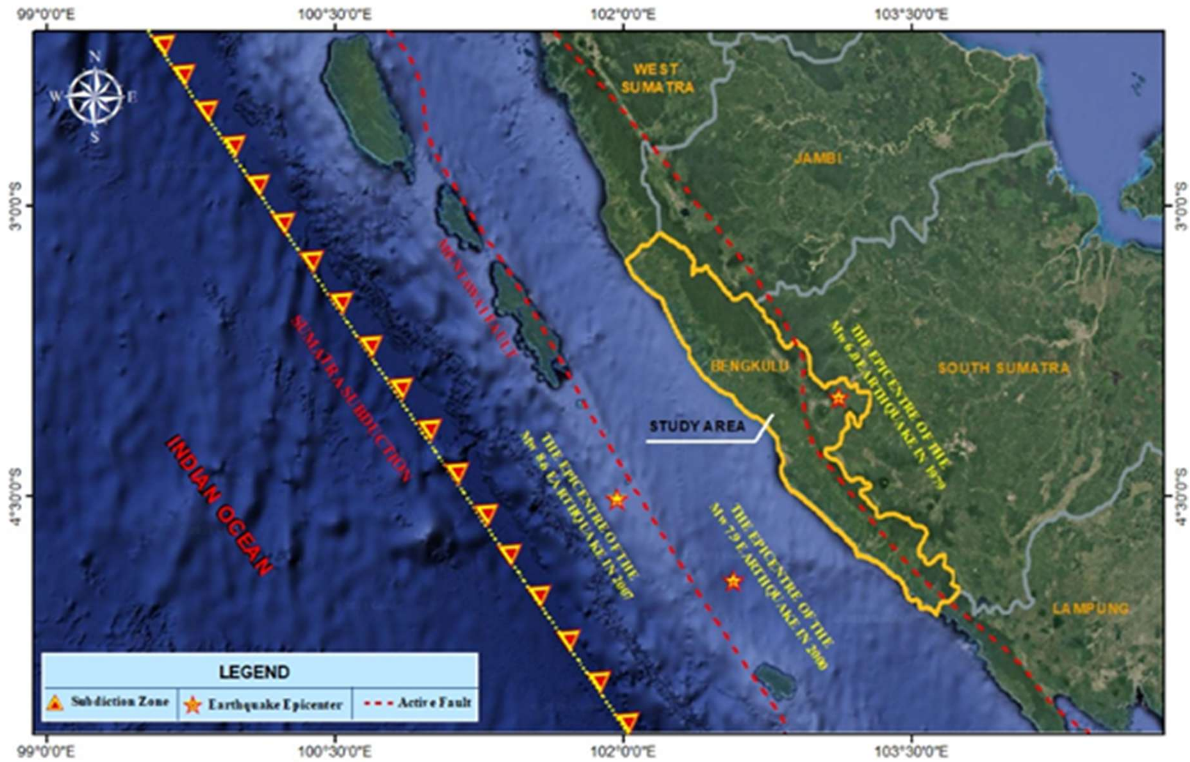

Figure 1. Seismotectonic Bengkulu Province

In the design and evaluation of earthquake-resistant buildings, soil-structure interaction is a crucial aspect because the dynamic response of a structure is influenced not only by the building's characteristics but also by the soil's dynamic behaviour. Therefore, an integrated approach between soil response analysis and structural analysis is necessary to obtain a more accurate estimate of seismic performance. However, most previous studies still analysed soil response and structural response separately, thus failing to represent the actual conditions of soil–structure interaction in evaluating the seismic performance of existing buildings, particularly educational buildings in earthquake-prone areas such as Bengkulu. This study analyses the seismic performance of the Faculty of Economics and Business building at Bengkulu University, which was evaluated using one-dimensional soil response analysis and dynamic structural analysis based on three strong earthquake records. Seismic hazard is one of the primary components that influence the estimation of the dynamic response of structures[5]. Earthquakes are disasters that cause significant damage to buildings or structures within the reach of seismic waves. Many building structures show vulnerability to seismic waves. Evaluating the seismic performance of educational buildings is crucial because they have high occupancy rates and serve as strategic public facilities, making structural failure due to earthquakes a significant safety risk. A structure is considered vulnerable if it cannot withstand seismic forces[6].

In earthquake-resistant structural design, buildings are constructed to remain intact and undamaged when faced with minor earthquakes that may occur over a shorter period[7]. Active tectonic plates cause areas near them to experience large earthquakes. With the increasing development in Bengkulu, particularly in educational buildings such as Bengkulu University, a structural performance analysis is necessary. The one-dimensional nonlinear soil response analysis

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was performed using DEEPSOIL, while the structural dynamic analysis was carried out using SAP2000. The novelty of this study lies in the integrated application of site-specific nonlinear soil response analysis and structural dynamic analysis to evaluate the seismic performance of an existing educational building in a highly seismic region of Indonesia. Unlike previous studies that primarily focus on either ground response or structural analysis, this study combines both approaches using three strong earthquake records to identify critical structural elements through stress-ratio evaluation. This integrated framework provides a more realistic assessment of building vulnerability and offers practical insights for improving earthquake-resistant design of educational facilities. Therefore, this study aims to evaluate the seismic performance of the Faculty of Economics and Business building at Bengkulu University through an integrated approach between site-specific soil response analysis and dynamic structural analysis.

2. Materials and Methods

2.1. Study Area

This section describes the location and general characteristics of the buildings under study. Location information is necessary to relate regional geological conditions and building characteristics to the seismic analysis that was conducted. The study was conducted at the Integrated Learning Building, Faculty of Economics and Business, University of Bengkulu, located in Bengkulu City. Figure 2 shows the layout of the research location used to determine the position of the building in relation to regional geological conditions. This information is important as a basis for determining seismic parameters and interpreting the influence of location conditions on earthquake response. This building serves as the centre of student activities and functions as a lecture hall. The three-dimensional model in Figure 3 is used as the basis for structural modelling in dynamic analysis. This model represents the geometric configuration of the building, the number of floors, and the distribution of structural elements, analysed using the finite element method. This building has three floors.

Figure 2. Layout of the study area

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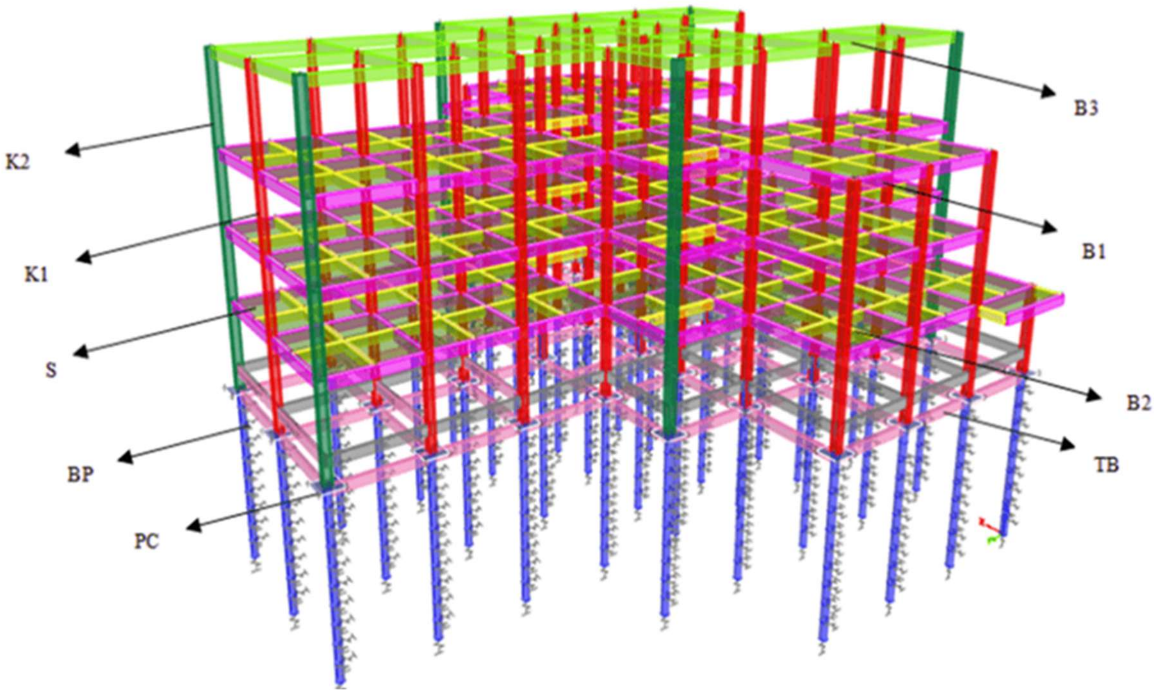

Figure 3. Three-dimensional modeling

2.2. Building Data and Characteristics

This section presents the geometric, material, and structural system data used in numerical modelling. This data serves as the basis for a dynamic structural analysis that represents the building's actual conditions. The Faculty of Economics and Business building at Bengkulu University measures 42 m in length, 40 m in width, and 13 m in height. The building structure is composed of reinforced concrete, with structural elements including columns, beams, ring beams, and others, as outlined in Table 1: Structural Element Data. From the table, details regarding structural elements, such as dimensions, thickness, and material quality, can be observed.

Table 1. Structure element data

No	Elements of structure	Notation	Dimension (mm)	Thickness (mm)
1	Bore Pile	BP	500	-
2	Tie Beam	TB	250×500	-
3	Sloof	S	250×500	-
4	Beam	B1	300×500	-
		B2	250×450	-
5	Beam Ring	B3	150×500	-
6	Column	K1	500×400	-
		K2	500×500	-
7	Slab		-	125
Material quality				
Concrete quality			fc'=18.675 MPa	
Reinforcement quality			Deformed bar	400 MPa
			Plain rebar	240 a

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2.3. Soil Data and Geotechnical Investigation

This section describes the soil data obtained from geotechnical investigations, which is used as the main input in soil response analysis. These soil parameters play an important role in determining the amplification of seismic motion from the bedrock to the surface. Figure 4 presents the soil profile resulting from geotechnical investigations, which is used as the main input in soil response analysis. Soil layer data, Vs values, and other dynamic parameters form the basis for calculating earthquake wave amplification from bedrock to the surface. There is a correlation between this earthquake and the size and length of structural damage expected. These three components of ground motion were used to obtain more accurate results and analysis, and several investigation data components were collected for a more comprehensive study, such as (N1)60, Vs, and PI data. There are three layers in the soil profile at the study site: a clay layer, a sandy layer, and a rock layer. From Figure 4, it can be seen that the soil data collected from around the relevant study site produced essential parameters such as corrected standard penetration test (N-SPT) values, shear wave velocity (Vs), unit weight (γ), and plasticity index (PI) obtained using a cone penetration test conducted to a depth of 9.6 m.

In comparison, the bedrock depth at the research site is approximately 33.9 m. Based on the figure, it is known that the first layer to the surface of the ground to a depth of 3.4 m is a clay layer of two sections, namely plastic clay (CH) and silty clay (CM), the second layer, sand, is located at depths of 6.4 to 9.6 m, sand layer, is composed of silt sand (SM) and well-graded sand (SW). The third one is the rock layer that is in a depth of 15 to 33.9 m.

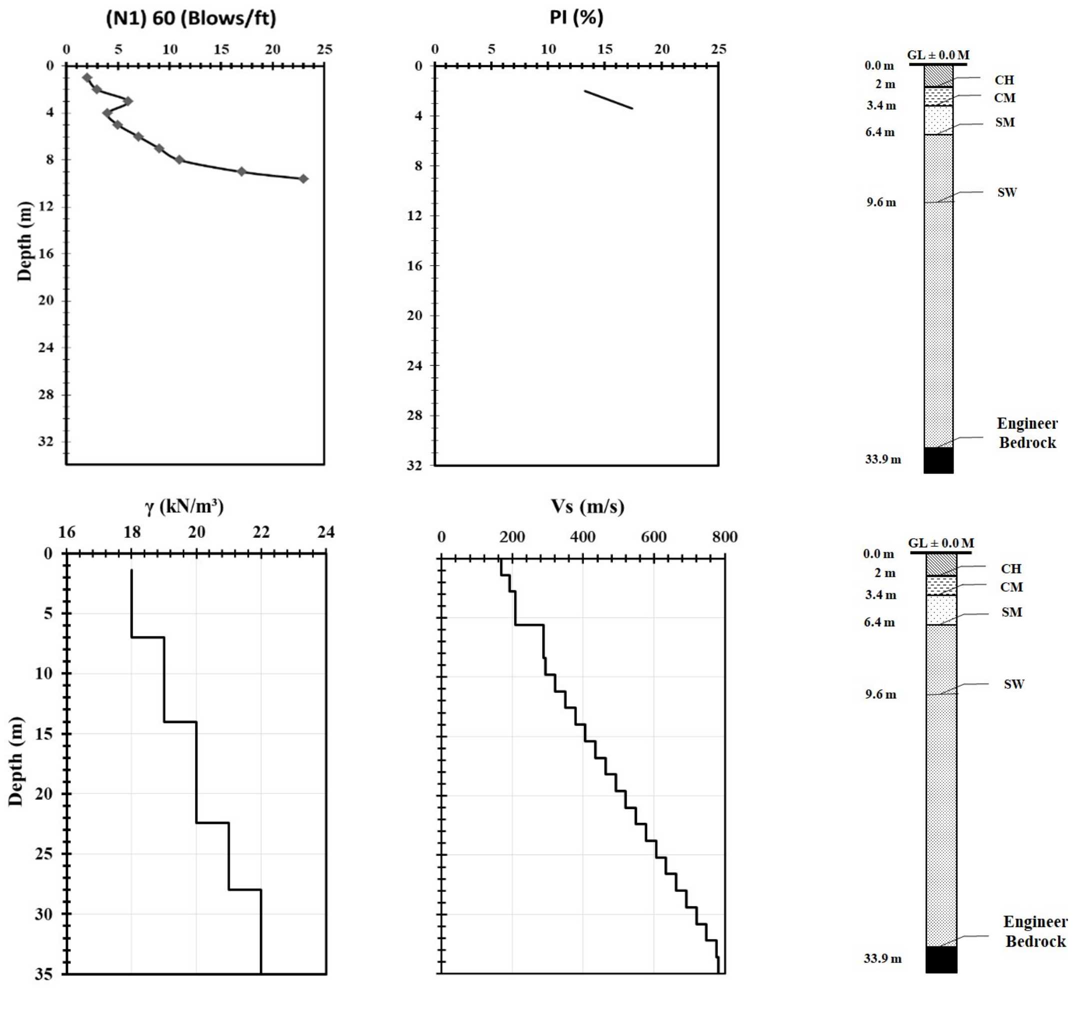

Figure 4. Geotechnical investigation data from the research site

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2.4. Combined analysis of ground motion and building dynamics

This section describes an integrated approach to soil response and structural dynamic analysis for evaluating the influence of local soil conditions on the seismic performance of buildings. When an earthquake occurs, structural performance is defined as a structure's ability to withstand applied loads without sustaining damage [2]. If a structure cannot withstand the load of an earthquake and its resulting impact, it is said to have failed. Analysing a building's performance is very important, especially when dynamic loads from earthquakes occur [8]. Structural performance during earthquakes is commonly evaluated using numerical approaches, such as the Finite Element Method (FEM), which allow the interaction between soil layers, foundations, and superstructures to be analysed [9]. In this study, ground response analysis was first conducted to propagate seismic waves from bedrock to the ground surface. The resulting surface motion was subsequently used as input for structural dynamic analysis, as illustrated in Figure 5.

Figure 5 illustrates the stages of the integrated analysis method used in this study. Earthquake waves are propagated from the bedrock to the surface through soil response analysis using the PDH model; the resulting ground motion at the surface is then used as input in FEM-based structural dynamic analysis. The Finite Element Method (FEM) is used in dynamic structural analysis because it can model the geometry of buildings and structural elements in detail. FEM enables dynamic analysis of structural response to seismic loads, including internal forces, deformations, and stress ratios in each structural element. The advantage of FEM over equivalent static methods is its ability to capture the overall distribution of structural response, thereby enabling better identification of critical elements that are prone to damage.

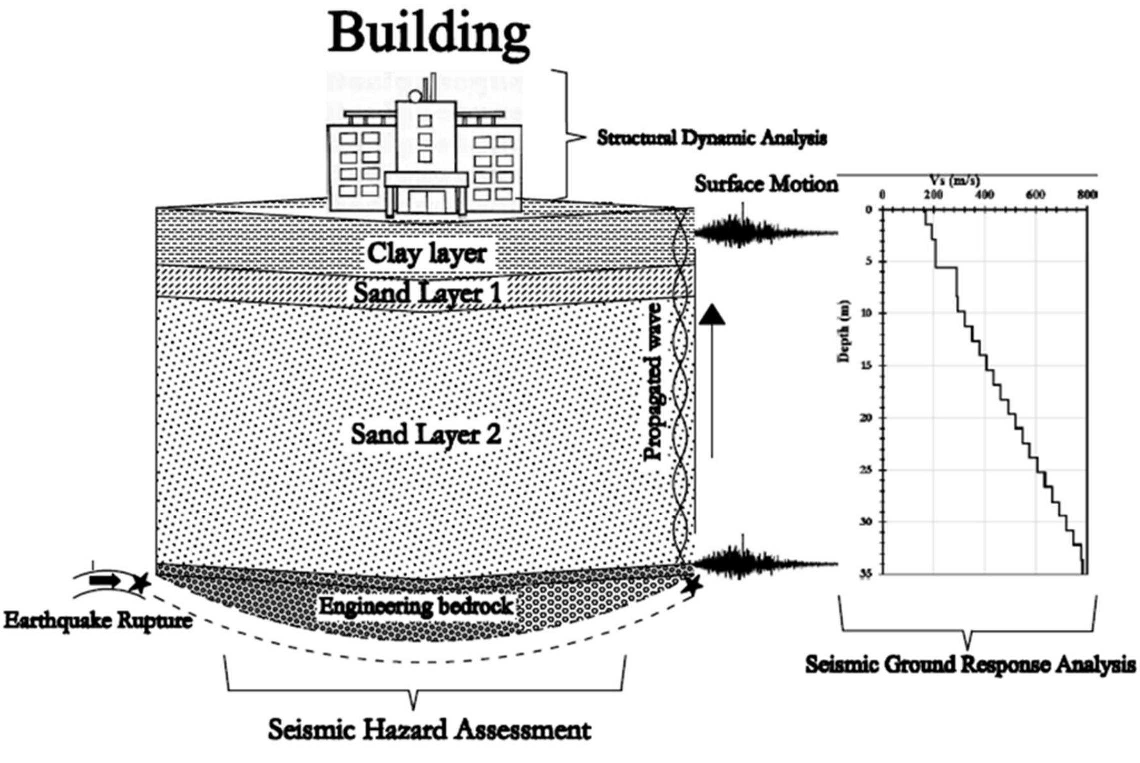

Figure 5. Illustration of the joint method for building assessment

The analysis begins by modelling the soil profile by inputting physical and technical soil parameters, as well as dynamic soil parameters. This study involved 25 soil layers, each having a depth/thickness (h) of 1.4 and a cumulative depth of 35.9. The waves generated by earthquakes travel to the ground surface through the bedrock layer of the Earth's crust. Soil layer information in this study was obtained from cone penetration tests conducted to a depth of 9.6 m, while the

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bedrock depth at the study site was estimated to be 33.9 m. The Vs value at a depth of 9.6 m was 288.573 m/s, and the Vs value at a depth of 33.9 m was 782.507 m/s. Subsequently, the bedrock parameters were input, namely the shear wave velocity (Vs), assumed to be 782.507 m/s², and the soil unit weight (γ).

2.5. Ground Motion Parameters

This section describes the ground motion parameters used in the analysis, such as acceleration, velocity, and ground displacement. These parameters are used to evaluate amplification levels and the effects of earthquakes on structures. Seismic waves are considered at each layer, although the surface soil at the bedrock surface is identified. The soil responses of the ground motion time histories are obtained by propagating through the soil. This is initiated by the outcomes of the one-dimensional nonlinear modelling carried out in the previous analysis phase using the Pressure-Dependent Hyperbolic (PDH) model. The Pressure-Dependent Hyperbolic (PDH) method was selected in the soil response analysis because it is capable of describing the nonlinear behaviour of soil due to seismic loads. This method accounts for changes in shear modulus and soil damping that depend on strain level, making it more realistic for soft-to-medium soil conditions, such as clay and sand layers at the study site. Compared to the linear method, PDH provides more accurate predictions in earthquake wave amplification and surface ground acceleration.

There is a correlation between this earthquake and the size and length of structural damage expected. These three components of ground motion were used to obtain more accurate results and analysis. Several investigation data components were collected for a more comprehensive study, such as (N1)60, Vs, and PI data[8].

The amplification factor (AF) is the ratio of the peak ground acceleration at the ground surface to that at the bedrock, reflecting the effect of local soil conditions on seismic wave propagation. Differences in wave-propagation velocity between the rock and the sediment strata are also a factor in this amplification. In general, Vs would be smaller at the surface than at the base of the rock due to the lower shear modulus (Gs) and damping factor, which affect the ground acceleration [10]. The amplification factor (AF) represents the change in ground acceleration as seismic waves propagate from bedrock to the ground surface and is defined as follows:

$$\mathrm{AFmax} = \frac{\mathrm{PGAsurface}}{\mathrm{PGAinput}} \quad (1)$$

Where AF is the amplification factor, $PGA_{surface}$ is the peak ground acceleration at the ground surface, and PGA_{input} the peak ground acceleration at the bedrock level. The amplification factor is calculated by comparing the PGA at bedrock and at the surface. Time histories of the pile tip layer and ground surface accelerations are other parameters that can be derived from ground response analysis. The highest acceleration values are plotted at these locations, and an amplification factor is obtained by comparing them[8].

2.6. Analysis Framework

This section presents the overall research methodology, from data collection to seismic performance evaluation of buildings. This framework ensures that each stage of analysis is carried out systematically and in an integrated manner. Figure 6 shows the overall research flow, from data collection and soil response analysis to dynamic structural analysis and seismic performance evaluation of buildings. This diagram clarifies the sequence of methodological steps applied in the study. This flow chart offers a scheme for examining the seismic performance of buildings in an integrated approach that considers not only site effects from soil response but also the influence of the building structure on in dynamics. It begins by defining the research problem and identifying the key needs by evaluating building performance under seismic loads in earthquake-prone regions. The data collection phase then gathers essential inputs, including site investigation data, earthquake ground-motion records, and detailed structural information for the buildings under study.

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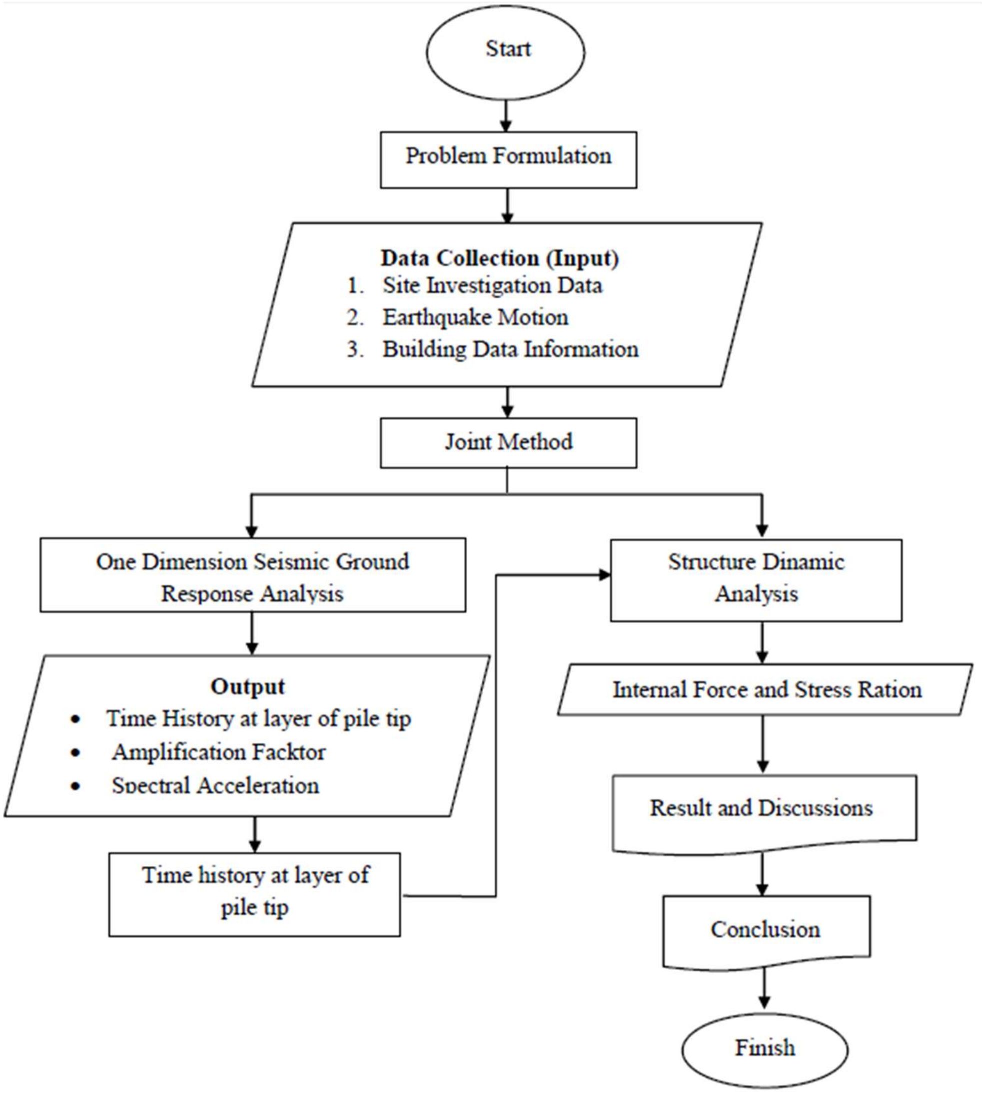

Figure 6. Research flowchart

The combined method underpins the analytical approach by integrating two primary components. The first component, one-dimensional seismic response analysis, propagates seismic waves from bedrock to the ground surface. This process generates outputs, including time histories at the pile tip, amplification factors, and spectral accelerations. These outputs capture modifications to the earthquake signal induced by local soil conditions, which are crucial for accurately representing ground motion. The second component, structural dynamic analysis, utilises the processed earthquake input to evaluate internal forces and stress ratios within structural elements. This analysis provides a quantitative assessment of structural loads resulting from ground vibrations. The results and discussion phase integrates outputs from both analyses by comparing internal force loads with structural capacity. This process identifies critical elements at risk of overstress and demonstrates the impact of soil-structure interaction on seismic performance. The conclusion synthesises these findings and underscores the need to combine geotechnical site response with dynamic structural modelling to improve building safety assessments. This methodological framework supports a comprehensive evaluation of earthquake vulnerability and informs strategies for earthquake-resistant design and retrofitting in high-risk areas.

This study combines seismic soil response methods and structural dynamics analysis by propagating the Mentawai, Kobe, and Coyote earthquake waves, which serve as the reference data

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for this research. The earthquake waves were propagated to the surface of the Faculty of Economics and Business building. The structural analysis aims to obtain information regarding structural safety through the stress ratio. Having carried out structural dynamic analysis, it is possible to discuss the structural behaviour in the case of an earthquake based on the internal forces experienced by the structure and the weakening of elements through stress ratios. If the stress ratio is red, it indicates that the stress ratio exceeds 1, signifying that the component is experiencing weakening due to excessive torsional and shear forces, resulting in overstress. Results and discussion.

3. Results and Discussions

The results of the one-dimensional nonlinear ground response analysis indicate that the peak ground acceleration (PGA) generally decreases with increasing depth. This behaviour is strongly influenced by the stratigraphy of the study area, where the upper soil layers consist mainly of clay deposits, followed by sandy layers, and finally bedrock at greater depths. The softer clay layer exhibits lower shear wave velocity (Vs), resulting in higher seismic wave amplification at the ground surface. This finding confirms that local soil conditions play a significant role in modifying earthquake ground motion characteristics before they reach the building foundation.

Among the three earthquake records used in this study, the Kobe earthquake produced the highest PGA at the ground surface, reaching approximately 0.941 g. In contrast, the Mentawai and Coyote earthquakes produced lower surface PGA values of approximately 0.245 g and 0.169 g, respectively. The significantly higher PGA observed during the Kobe earthquake can be attributed to its shallow crustal source mechanism and high-frequency content, which tend to generate stronger surface acceleration demands than subduction earthquakes. This result aligns with previous studies, which found that shallow crustal earthquakes typically generate higher PGA values and more severe structural impacts due to shorter travel distances and less attenuation of seismic energy.

The amplification observed in the clay layer supports the findings of [1], who reported that soft clay deposits in Bengkulu tend to increase surface acceleration due to the contrast in shear modulus between soil layers and bedrock. Similar conclusions were also reported in several site response studies in seismic regions, indicating that soft soil layers can significantly increase seismic demand and structural vulnerability. Therefore, the obtained PGA results confirm that the FEB building is located in a site condition where soil amplification may increase the risk of structural damage during strong earthquakes.

3.1. PGA (Peak Ground Acceleration)

Peak Ground Acceleration (PGA) is the maximum ground acceleration due to the propagation of seismic waves, while ground amplification describes the increase in acceleration response from the bedrock to the surface resulting from differences in the dynamic properties of soil layers. Figures 7(a), (b), and (c) present peak ground acceleration (PGA) profiles for the Mentawai, Kobe, and Coyote earthquakes. Percepatan gempa menurun seiring bertambahnya kedalaman tanah. Hal ini sesuai dengan stratigrafi lokasi penelitian, di mana lapisan atas berupa lempung dan berubah menjadi pasir pada kedalaman yang lebih besar. At 33.9 metres, the presence of rock further reduces PGA values compared to the surface. Variasi ini terjadi akibat faktor amplifikasi lokal dan perbedaan kecepatan gelombang geser (Vs) pada setiap lapisan tanah[11]. These results indicate that local soil conditions play a significant role in amplifying ground motion. Amplification factors describe the increase in soil movement acceleration as it propagates from the bedrock to the surface. Perbedaan nilai Vs antara batuan dasar dan tanah permukaan menyebabkan peningkatan amplifikasi gelombang gempa[12]. Soil movement amplification factors can reflect the level of significant soil movement fluctuations that occur during seismic wave propagation. This indicates that amplification factors are strongly influenced by soil classification[13].

At the research site, the clay layer showed the highest amplification compared to the sand layer and bedrock, which directly contributed to the increase in PGA and PGV at ground level.

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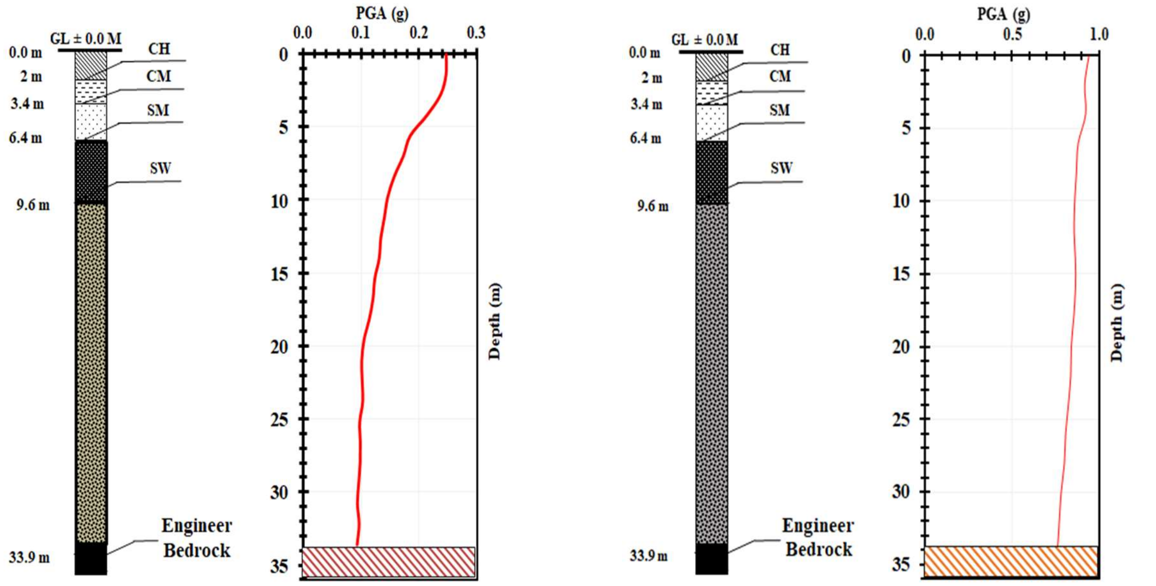
Amplification factors are influenced by soil type, and amplification values tend to increase in clay layers[14]. The pattern of decreasing PGA values with increasing depth indicates the significant role of soil conditions in earthquake response. The clay layer at the surface produces the highest amplification, indicating that buildings in this location experience greater seismic loads than if built on stiffer soil. These findings confirm that local soil conditions directly contribute to an increased risk of structural damage, especially in earthquakes with shallow sources such as the Kobe earthquake. Overall, the PGA values from the analysis indicate that the ground acceleration response at the surface remains within the range anticipated in earthquake planning per SNI 1726:2019. Although there is significant amplification in the clay layer, the structural design has accounted for the effect of this maximum acceleration. Therefore, in terms of PGA parameters, the building generally still meets earthquake resistance requirements, although locations with soft soil require special attention

Peak Ground Acceleration (PGA) is the highest ground acceleration recorded in an area due to seismic wave propagation[15]. Peak ground acceleration (PGA) and vibration periods at the study site may be related to earthquake vibrations[16]. The peak structural response typically occurs shortly after the peak ground acceleration (PGA) is reached. This delay arises because structures require a brief period to overcome inertial resistance before responding to earthquake-induced forces[17]. Figure 7 illustrates that the maximum peak ground acceleration (PGA) for the Mentawai earthquake occurred at ground level in clay soil, with a value of approximately 0.245 g. At a depth of 6.4 m, corresponding to a mud (SM) layer, the PGA decreased to about 0.182 g. At 9.6 m, within well-graded sand (SW), the PGA was approximately 0.145 g. For the Kobe earthquake, the maximum PGA at the ground surface in clay soil was approximately 0.941 g. At 6.4 m in mud (SM), the PGA was about 0.871 g, and at 9.6 m in well-graded sand (SW), it was approximately 0.855 g. The Coyote earthquake exhibited a maximum PGA of 0.169 g at the ground surface in clay soil, 0.142 g at 6.4 m in silt (SM), and 0.137 g at 9.6 m in well-graded sand (SW).



(a) Mentawai (2007)

(b) Kobe (1995)

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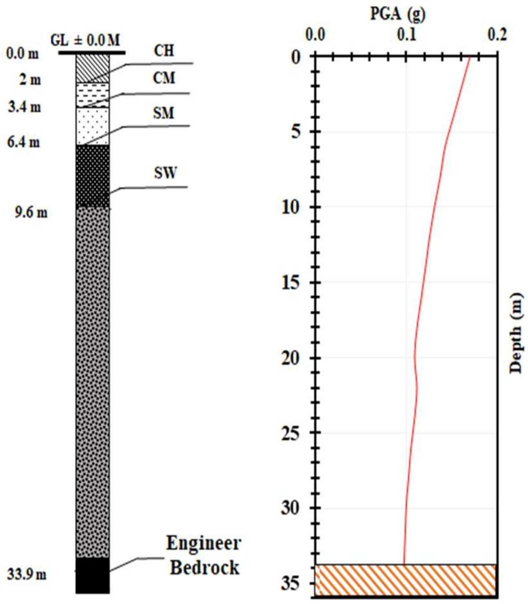

(c) Coyote (1979)

Figure 7. Peak Ground Acceleration Profile of Ground Response Analyses

These results indicate that PGA is strongly influenced by soil classification, with higher values observed in clay soils. This is due to several factors, one of which is that the epicentre of the Kobe earthquake was shallower than that of the Mentawai and Coyote earthquakes, which were relatively deeper. This is why the PGA values for the Kobe earthquake are so high: a shallow epicentre makes the quake felt directly at the surface, thereby significantly affecting surface damage to structures [18]. This event is noted as one of the most powerful earthquakes in modern Japanese history.

The ground surface time history is then used to base the structural analysis and determine the structure's response to the relevant time history. Some researchers have studied the attenuation of earthquake ground motion (its time behavior), and the structural assessment will be evaluated [19]. The time-history dynamic analysis method is utilised to analyse the earthquake acceleration response of structures. The earthquake accelerogram used in time-history dynamic response analysis must be derived from recordings of ground motion caused by earthquakes at locations with geological, topographical, and seismotectonic conditions similar to those of the area being analysed [20]. Figure 8 shows the PGA values for the Mentawai, Kobe, and Coyote earthquakes. The highest PGA value of the Mentawai earthquake was obtained on the surface layer with the value of 0.245 g, the greatest PGA value of the Kobe earthquake was obtained on the surface layer with the value of 0.941 g, and the greatest PGA value of the Coyote earthquake was obtained on the surface layer with the value of 0.169 g. The ground acceleration history in Figure 8 shows that the Kobe earthquake had a greater amplitude and duration of vibration than the Mentawai and Coyote earthquakes. This condition indicates that the building structure experienced higher and repeated inertial loads, thereby increasing the potential for structural damage accumulation. The relatively long vibration duration can also increase the structure's dynamic response, especially in beam and column elements.

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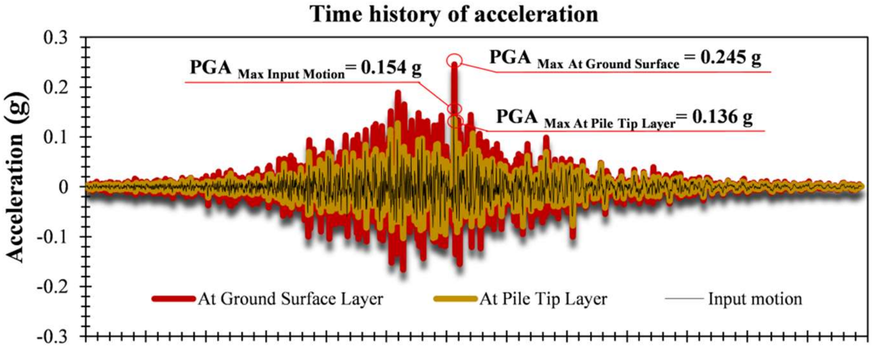
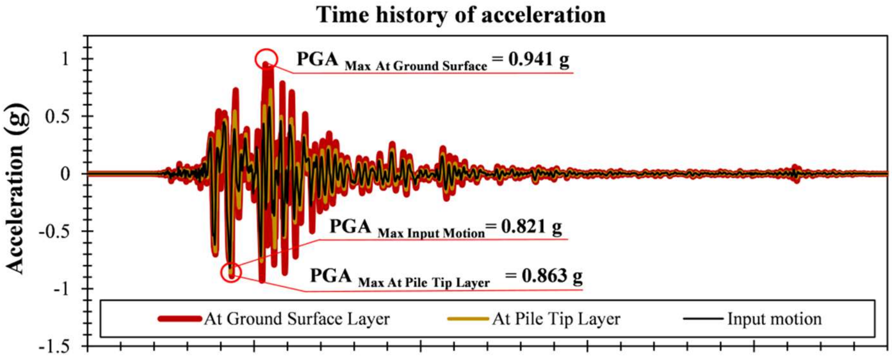
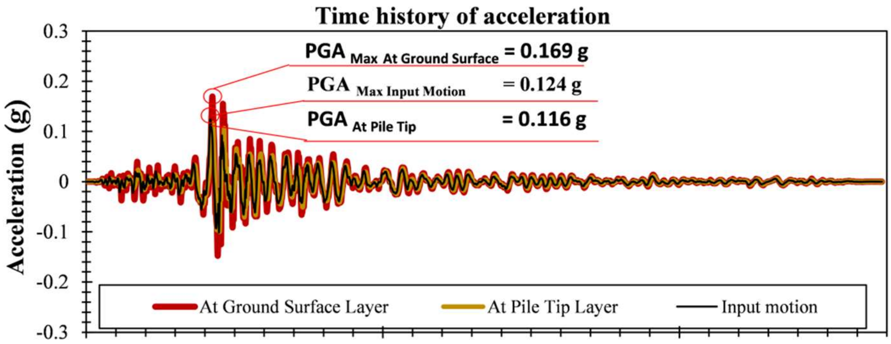


(a) Mentawai (2007)



(b) Kobe (1995)



(c) Coyote (1979)

Figure 8. History of ground motions recorded at ground surface and pile tip layer, and input motions to acceleration (a)Mentawai (b)Kobe (c) Coyote

3.2. Peak Ground Velocity (PGV)

The PGV values for the Mentawai, Kobe, and Coyote earthquakes are also provided in Figure 9. PGV analysis shows that the Kobe earthquake produced the highest surface-layer ground velocity values among the Mentawai and Coyote earthquakes. These high PGV values reflect the large amount of seismic energy and the potential for increased deformation demands on building structures. The best value of the P G V of the Mentawai earthquake wave was realised at the surface layer of 0.168 g. The maximum PGV was observed in the topmost layer during the Kobe earthquake, with a value of 1.057 g. The maximum PGV of the Coyote earthquake wave was measured in the surface layer, producing a value of 0.022 g.

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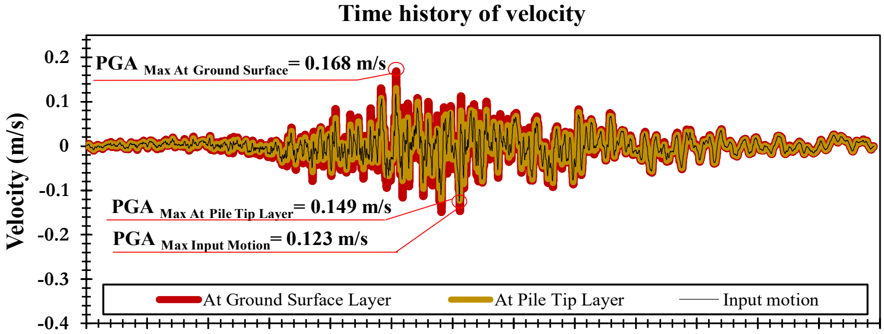
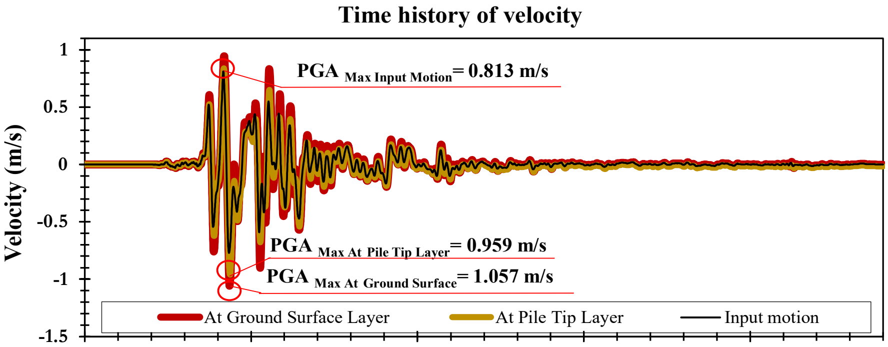
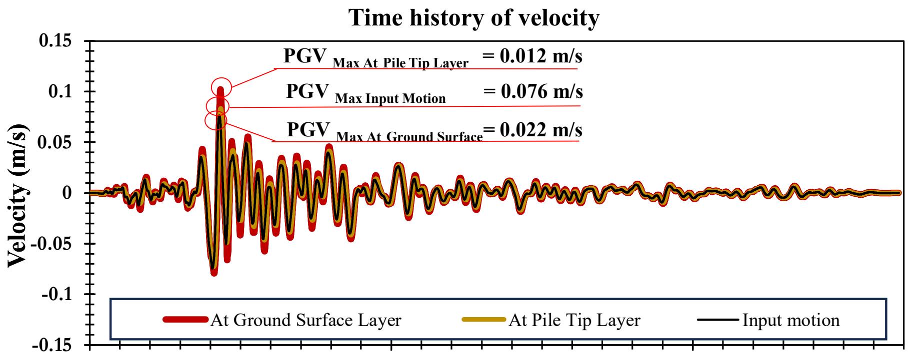


(a) Mentawai (2007)



(b) Kobe (1995)



(c) Coyote (1979)

Figure 9. Time histories of ground motions at ground surface, pile tip layer and input motion for velocity (a)Mentawai (b)Kobe (c) Coyote

High PGV values, particularly in the Kobe earthquake, indicate increased deformation demands on structures. PGV is closely related to the potential for nonlinear structural damage, so high PGV values in the surface layer indicate a greater risk of damage to building structural elements. This explains why the Kobe earthquake produced a more critical structural response than the other two earthquakes. The PGV analysis shows that the Kobe earthquake had the highest deformation demand among the three earthquakes. However, the PGV values obtained are still in line with the assumptions of dynamic structural design in accordance with SNI 1726:2019. This indicates that, overall, the building design remains adequate, although increased ground velocity could increase the risk of local damage to certain elements.

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3.3. Peak Ground Displacement (PGD)

Figure 10 shows the PGD values for the Mentawai, Kobe, and Coyote earthquake waves. The maximum PGD value associated with the Mentawai earthquake wave was observed in the surface layer, with a moving and input motion of 0.097 g. The maximum PGD of the Kobe earthquake wave was measured in the pile tip layer and was 0.706 g. The maximum PGD value recorded by Coyote was in the surface layer, at 0.012 g.

A higher maximum ground displacement (PGD) indicates the potential for significant ground deformation during an earthquake. High PGD values, particularly in the Kobe earthquake, indicate the possibility of relative displacement between the ground and structures, which can increase the deformation demands on foundation systems and substructure elements. This condition has the potential to accelerate structural damage if not anticipated in the design planning.

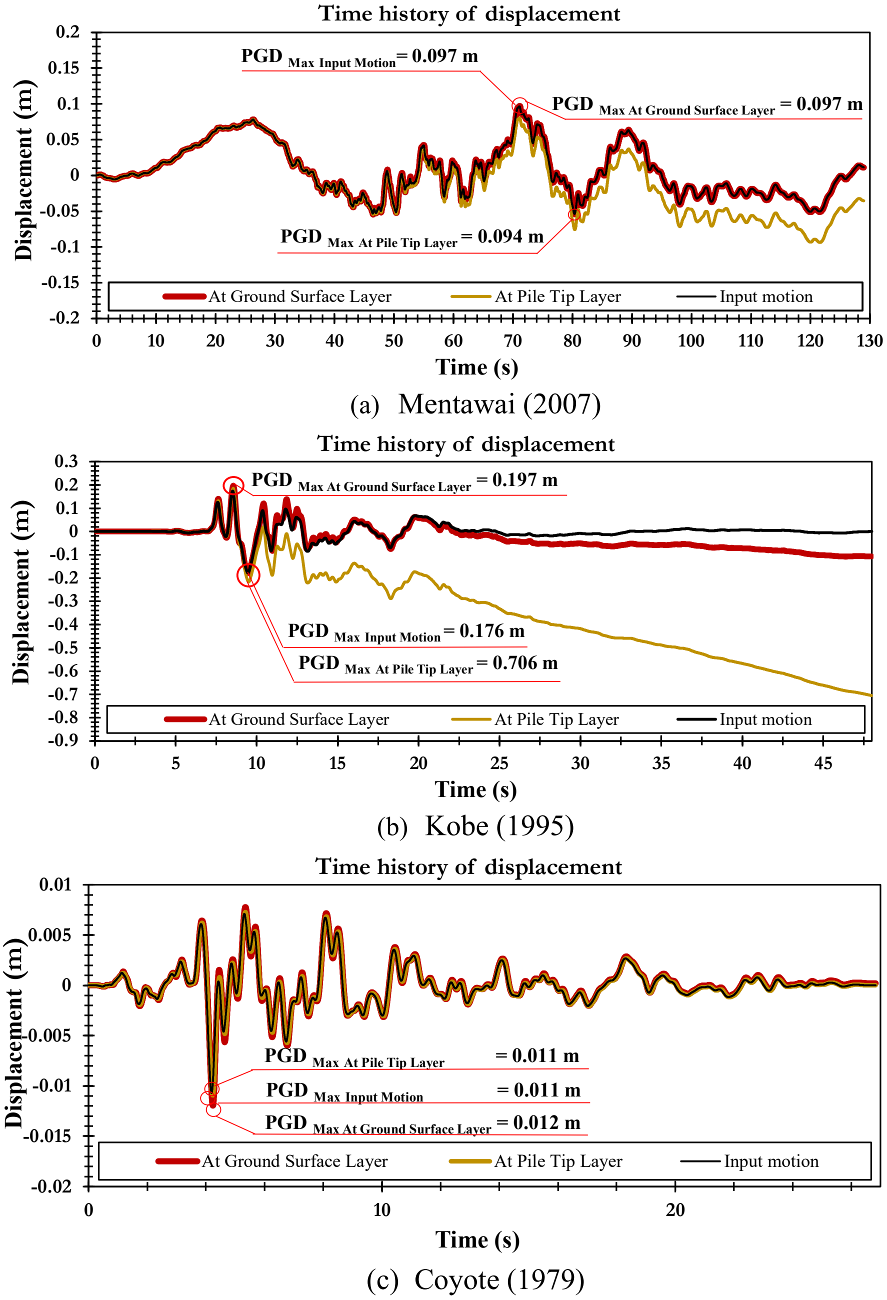

Figure 10. Ground motion record at the ground surface, pile tip layer, and input motion to the displacement (a) Mentawai (b) Kobe (c) Coyote

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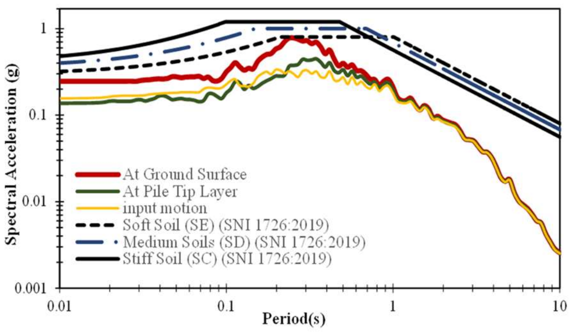
3.4. Spectral Acceleration Response

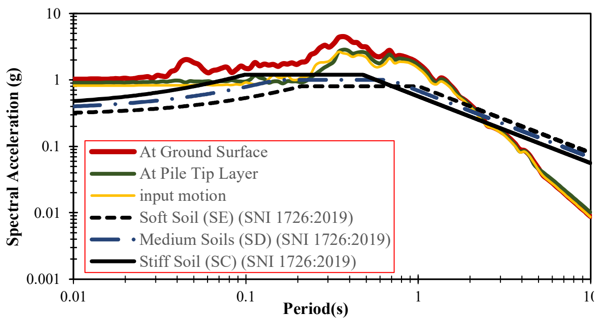
Figure 11 compares the spectral acceleration at the ground surface, at the pile tip, and in the input motion. This analysis compares the spectral acceleration at the research location with the designed spectral acceleration[11]. This analysis process is intended to estimate spectral acceleration parameters, which are used as a reference in creating representative soil-motion models for each study location [8]. The spectral acceleration values produced vary depending on the strength of the earthquake waves and differ in each period. The acceleration spectrum analysis in Bengkulu City, based on SNI 1726:2019, the latest earthquake design code, shows that the lowest spectral accelerations occur in the input waves and foundation layers. This indicates that the structural design of the FEB Building at Bengkulu University has considered the strength and resilience required to withstand earthquakes. This study shows that the building meets earthquake design code standards and provides relatively safe results during earthquakes. The acceleration spectrum curve was compared with the design acceleration spectrum listed in SNI 1726:2019.

The comparison of spectral accelerations in Figure 11 shows that the spectral response at ground level is higher than that at the foundation layer and input motion. This indicates that local soil conditions amplify the seismic response at certain periods that may resonate with the building's natural period. The mismatch between the soil's dominant period and the structure's period can increase the building's dynamic response and magnify the internal forces acting on the structural elements. From the soil response analysis of the Mentawai earthquake waves, the maximum spectral value at the ground surface is 0.781 g, occurring at a period of 0.253 s. The maximum spectral value for the pile tip layer is 0.448 g, which occurs at a period of 0.346 s. At the same time, for input motion, the maximum spectral value is 0.335 g, which appears at a period of 0.198 s.

Based on soil response analysis to the Kobe earthquake, the acceleration spectrum analysis for Bengkulu City (SNI 1726:2019) shows higher spectral values than the maximum spectral values at the ground surface, pile tip, and input motion. Through the study of the Coyote earthquake wave propagating to the ground surface, can be seen in the Ground response spectral shows a maximum value of 0.507 g at a period of 0.416 s. The most extreme spectral measure for the pile tip layer is 0.422 g as observed at a time period of 0.416 s. At the same time, for the input motion, the biggest spectral value 0.399 g is served at the period of 0.417 s. A comparison of the analysis results' spectral response with the design spectrum of SNI 1726:2019 shows that the design curve is above the actual spectral response at the building's dominant period. This condition confirms that the structural design has incorporated sufficient safety factors for the planned seismic load. Therefore, in terms of spectral response, the building can be categorised as meeting seismic design adequacy.



(a) Mentawai (2007)



(b) Kobe (1995)

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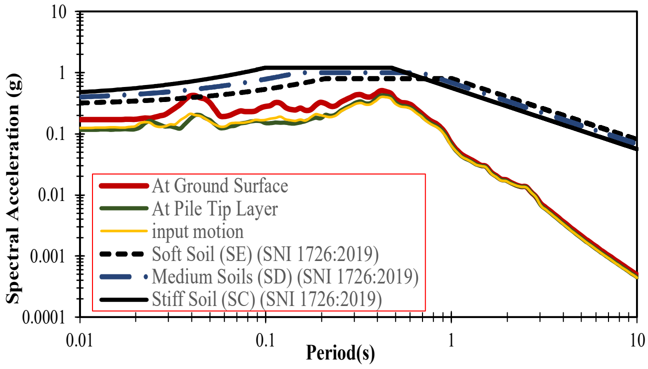

(a) Coyote (1979)

Figure 11. Spectral acceleration comparison (a)Mentawai (b)Kobe (c) Coyote

3.5. Structural Dynamic Response

Tables 2, 3, and 4 present the maximum moment forces before and after the application of the Mentawai, Kobe, and Coyote earthquake loads, along with the percentage increase from before and after the earthquake loads were applied. Regular forces, normal forces, and moments vary depending on the magnitude of the earthquake. From Tables 2, 3, and 4, it is evident that internal forces in the structure increased between the pre- and post-earthquake load application periods.

Table 2. Structural style assessment before and after the Mentawai earthquake

Element	Dimension (cm)	Before Earthquake Load is Applied			After Earthquake Load Applied Mentawai			Percentage Increase		
		Normal force (kN)	Shear force (kN)	Bending moment (kNm)	Normal force (kN)	Shear force (kN)	Bending moment (kNm)	Normal force (%)	Shear force (%)	Bending moment (%)
Column 1	400 × 500	0.307	54.182	97.535	0.366	91.089	163.456	19	68	68
Column 2	500 × 500	0.239	80.487	125.874	0.246	164.901	242.649	3	105	93
Beam Ring	150 × 500	12.82	16.296	19.436	24.294	28.654	24.409	90	76	26
Beam 1	250 × 450	3.495	26.799	28.185	16.912	124.357	92.588	384	364	229
Beam 2	300 × 500	27.528	87.653	80.124	55.048	153.200	143.687	100	75	79
Sloof	250 × 500	31.918	101.493	155.879	71.035	183.378	290.164	123	81	86
Tie Beam	250 × 500	58.733	126.866	194.848	87.499	229.222	362.706	49	81	86

Table 3. Structural style assessment before and after the Kobe earthquake

Element	Dimension (cm)	Before Earthquake Load is Applied			After Earthquake Load Applied Kobe			Percentage Increase		
		Normal force (kN)	Shear force (kN)	Bending moment (kNm)	Normal force (kN)	Shear force (kN)	Bending moment (kNm)	Normal force (%)	Shear force (%)	Bending moment (%)
Column 1	400 × 500	0.307	54.182	97.535	0.418	124.950	201.735	36	131	107
Column 2	500 × 500	0.239	80.487	125.874	0.289	198.772	296.884	21	147	136
Beam Ring	150 × 500	12.82	16.296	19.436	29.918	38.644	36.784	133	137	89
Beam 1	250 × 450	3.495	26.799	28.185	9.274	124.357	92.588	235	82	112
Beam 2	300 × 500	27.528	87.653	80.124	69.441	198.507	176.983	152	126	121
Sloof	250 × 500	31.918	101.493	155.879	89.663	228.911	361.552	181	126	132
Tie Beam	250 × 500	58.733	126.866	194.848	112.804	287.346	459.118	92	126	136

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Table 4. Structural style assessment before and after the Coyote earthquake

Element	Dimension (cm)	Before Earthquake Load is Applied			After Earthquake Load Applied			Percentage Increase		
					Coyote					
		Normal force (kN)	Shear force (kN)	Bending moment (kNm)	Normal force (kN)	Shear force (kN)	Bending moment (kNm)	Normal force (%)	Shear force (%)	Bending moment (%)
Column 1	400 × 500	0.307	54.182	97.535	0.309	67.842	114.206	0	25	17
Column 2	500 × 500	0.239	80.487	125.874	0.236	101.907	149.337	1	27	19
Beam Ring	150 × 500	12.82	16.296	19.436	16.883	21.276	24.108	32	31	24
Beam 1	250 × 450	3.495	26.799	28.185	7.326	56.298	55.148	110	110	96
Beam 2	300 × 500	27.528	87.653	80.124	39.206	11.604	108.529	42	87	35
Sloof	250 × 500	31.918	101.493	155.879	46.811	129.388	198.441	47	27	27
Tie Beam	250 × 500	58.733	126.866	194.848	71.003	161.212	246.922	21	27	27

According to the calculation based on the earthquake waves in Mentawai, it is apparent that beam element 1 saw an increase in normal force by 384%, shear force by 364%, and bending moment by 229%. These are shown in Table 2. Moreover, according to the results obtained in the behavior of the Kobe earthquake wave behavior analysis, the normal force experienced by beam element 1 increased by 235%, the shear force by 82%, and the bending moment by 88%, as indicated in Table 3. Subsequently, based on the analysis of the Coyote earthquake waves, beam element 1 was found to undergo an increase in normal force of 110%, shear force of 110%, and bending moment of 96%, as depicted in Table 3. This is because beam one is in a very critical position during an earthquake. The stress ratio distribution in Figure 12 shows that the beam element (Beam 1) is the most vulnerable structural component to seismic loads. The occurrence of a stress ratio greater than one on the second and third floors indicates that this element exceeded its design capacity during the strong earthquake. The concentration of critical elements at the rear and sides of the building indicates an imbalance in the distribution of internal forces, which could potentially be the starting point for structural failure in the event of a more intense earthquake. Unsafe structural elements are marked in red, specifically beam 1 (B1) on the second and third floors. On the second floor, there are two unsafe beams located at the rear of the building. On the third floor, eight unsafe beams or elements are experiencing structural weakening at the rear and sides of the building.

Beam 1 was identified as the weakest structural element due to a combination of several technical factors. Geometrically, Beam 1 has a relatively smaller cross-sectional dimension compared to other beam elements, resulting in more limited flexural and shear capacity. In addition, Beam 1 is located in the zone of the building that receives a greater concentration of seismic forces due to the uneven distribution of mass and structural stiffness, particularly on the second and third floors. This condition causes Beam 1 to experience a significant increase in axial force, shear, and bending moment during earthquake loading. In terms of location, Beam 1 serves as the main connecting element at the rear and sides of the building, which plays a role in transferring lateral forces from earthquakes to the vertical elements. Under these conditions, Beam 1 functions as part of the moment-resisting frame system, thereby experiencing greater deformation demands compared to secondary beams. This explains the high stress ratio observed in Beam 1 element after the earthquake load was applied. In addition to geometric and location factors, the increase in dynamic soil response due to amplification in the clay layer also increases the earthquake load received by the structure. The inertial load from high soil acceleration significantly increases lateral forces, making Beam 1 more vulnerable because it is located along the structure's main load path. This condition causes Beam 1 to reach or exceed its design capacity faster than other structural elements. Thus, Beam 1 can be categorised as a critical element that requires special attention in seismic performance evaluation and future structural reinforcement planning.

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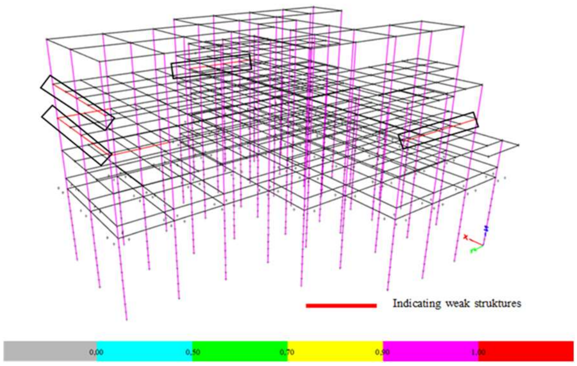
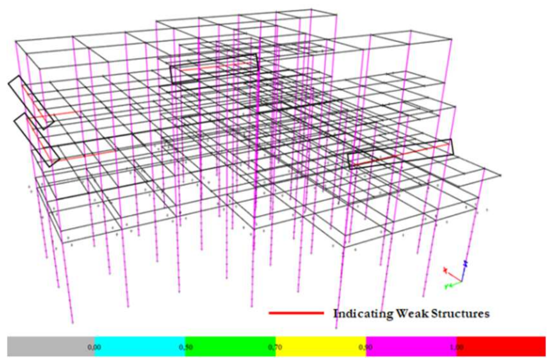
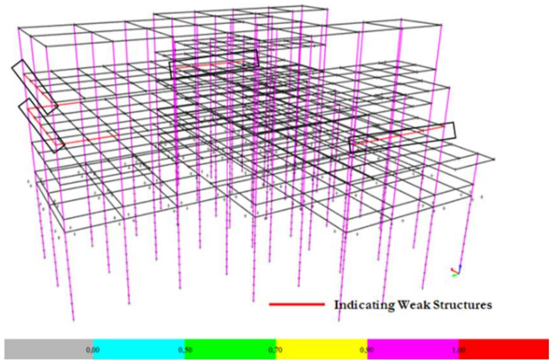
The stress ratio analysis successfully identifies critical structural components requiring retrofitting, particularly Beam 1 elements on the second and third floors, where the stress ratio exceeds unity under strong earthquake excitation. The quantified increases in PGA, PGV, spectral acceleration, and structural stress ratios provide a data-driven justification for enhancing seismic resilience in educational facilities, particularly for buildings located on soft soil deposits. Dynamic structural response analysis shows that although the building globally complies with the earthquake design requirements of SNI 1726:2019, some structural elements exhibit responses close to or exceeding their design capacity. This indicates that the design adequacy has been generally met, but there are critical elements that require evaluation and potential reinforcement to improve the building's seismic performance.



(a) Mentawai (2007)



(b) Kobe (1995)



(c) Coyote (1979)

Figure 12. stress ratio of concrete structure (a)Mentawai (b)Kobe (c) Coyote

4. Conclusions

The study finds that although the Faculty of Economics and Business building at Bengkulu University generally meets seismic design requirements, soil amplification, particularly within clay layers, significantly increases surface PGA and increases vulnerability in certain structural elements, especially Beam 1, under strong ground motion. The integrated analysis of site-specific soil response and structural dynamics effectively identified seismic vulnerabilities of the Faculty of Economics and Business building at Bengkulu University. The Kobe earthquake scenario produced the most severe impact, with a maximum PGA of 0.941 g and critical surface amplification, causing excessive forces in key structural components, particularly Beam 1. These results highlight the importance of the proposed integrated framework for accurate seismic vulnerability assessment and strengthening of educational buildings in high-risk seismic areas. The novelty of this research lies in the application of a combined approach between soil response analysis and dynamic structural analysis in assessing the seismic performance of educational buildings in earthquake-prone areas. This approach allows the influence of local soil conditions on structural response to be directly accounted for, making the identification of critical structural elements more accurate than

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in conventional structural analysis. Thus, this method provides practical benefits for evaluating and planning the reinforcement of educational buildings to make them more resilient to earthquakes in active seismic zones. As a follow-up to the findings of this study, it is recommended that an evaluation and reinforcement of critical structural elements be conducted, particularly Beam 1 on the second and third floors, which showed a stress ratio exceeding its design capacity. Reinforcement measures that can be considered include increasing flexural and shear capacity through the addition of reinforcement or jacketing methods, as well as reviewing the distribution of structural stiffness to reduce the concentration of seismic forces. In addition, the application of local reinforcement strategies and re-evaluation of structural design based on dynamic analysis assisted by ground response is highly recommended to improve the seismic performance of educational buildings in earthquake-prone areas.

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