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Preliminary communication

Numerical analyses to observe the performance of a monumental building at the University of Bengkulu, Indonesia

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Abstract

The Integrated Laboratory Building of Political and Social Science Faculty, located in a seismic-prone area, has been the subject of a comprehensive study of its performance using seismic response analysis. This research has yielded significant findings, gathering crucial secondary data, such as soil layers, bedrock depth, building structure, and earthquake wave information. The seismic response analysis, conducted by referencing the synthetic earthquake wave from the Bengkulu–Mentawai earthquake in 2007, with a magnitude of M_w 8.6, has provided a comprehensive overview of the earthquake waves at the foundation soil layers. The data, analysed using the finite element method to understand the structural response, revealed that the Peak Ground Acceleration (PGA) and amplification factor at the surface soil layer are 0.220g and 1.429, respectively. The most critical PGA and amplification factors at the foundation soil layer are 0.147g and 0.955. Structural analysis has revealed internal forces and beam elements experiencing over-strength, necessitating retrofitting the affected structural elements to reduce the impact. One practical and highly effective method of retrofitting involves increasing the beam dimensions by 53.12%. With retrofitting, the impact of structural deformation can be minimised, enhancing the building's resilience in case of an earthquake of equal or greater magnitude. These findings underscore the importance of our research and highlight the significant role of engineers, architects, and researchers in ensuring the safety and longevity of structures in seismic-prone areas.

Keywords:
amplification factor; building performance; PGA; seismic response; retrofitting

1. Introduction

Bengkulu Province, situated in an earthquake-prone zone comprising two strike-slip faults and a subduction zone known as the Mentawai fault, the Sumatra fault, and the Great Sumatra Subduction (**Mase et al., 2021a**), has a history of seismic activities (see **Figure 1**). The Bengkulu-Mentawai earthquake, a severe earthquake that occurred on September 12, 2007, with a magnitude of M_w 8.6, was one of the most devastating earthquakes to hit Bengkulu city in the past two decades (**Mase, 2017**). This highly destructive earthquake, triggered by Sumatra subduction activity, resulted in 25 fatalities, 161 injuries, and damage to 56,425 buildings, including residential, public, and government infrastructures (**Meteorological, Climatological, and Geophysical Agency, 2019**). The impact of such earthquakes on structures is significant and requires immediate inspection, especially in developing cities like Bengkulu. **Misliniyati et al. (2018)** have underscored the immediate need for seismic hazard assessment for structural design and spatial regional development to minimise potential damage, highlighting the situation's urgency.

The existence of the University of Bengkulu is vital for the local population's higher education. As such, efforts continue to improve, with several new facilities and infrastructure enhancements being constructed. One new facility was established in 2021 as an integrated laboratory for the Political and Social Science Faculty. This new building acts as the faculty's symbol and is one of the supporting buildings for students carrying out practical activities. In addition, the building is used as a laboratory, where many students visit. Therefore, inspecting this building, especially its structural performance, is necessary.

Building performance is the ability of a structure to maintain its nonlinear behaviour when subjected to seismic loads at a certain level without experiencing severe damage under an earthquake (**Filiatrault and Sullivan, 2014; Yadhav et al., 2024**). An integrated approach can be used to investigate building performance. The integrated approach combines ground response analysis and structural analysis. The ground response analysis is used

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to obtain a site-specific response. The output from the ground response analysis is then used to check the structural response under dynamic analysis. The method that can be implemented to perform ground response analysis has been introduced by many researchers, such as **Likitlersuang et al. (2020)**, **Qodri et al. (2021)**, and **Somantri et al. (2023)**. The concept is to propagate seismic waves and to obtain fundamental results, such as soil response, frequency content, and time history data. These results are then implemented to simulate structural dynamic analysis.

The structural dynamic analysis for building performance is a priority when responding to dynamic problems such as earthquakes (**Muin and Mosalam, 2021; Freddi et al., 2021**). Such analysis can be carried out using the finite element method (FEM) for structural analysis, which is integrated into the code of three-dimensional finite element analysis in Extended Three-dimensional Analysis of Building Systems (https://www.csiamerica.com/products/etabs/trial). FEM applies numerical analysis, which can then be used to analyse the response of structures subjected to earthquake loads (**Hall, 2003; Liu et al., 2020**). FEM can also be used for interactions between the superstructure of a building, the foundation, and the soil on which the building stands (**Kumar et al., 2020; Huynh et al., 2022**). FEM is later combined with the result of the seismic response analysis. The seismic response analysis is carried out by propagating a one-dimensional wave into the soil layers under the building, producing parameters for ground movements caused by earthquakes (**Mase et al., 2022a**). These ground movement parameters are later used to analyse structural performance using time history data and FEM. The results in terms of the distribution of internal forces and stress ratio could show the actual performance of a building during a potential earthquake.

In Bengkulu city, a region with high seismic activity, it is imperative that any building construction strictly adheres to the approved seismic design code **SNI 1726:2019 (2019)**, an updated version of **SNI 1726:2012 (2012)**. This code is based on a thorough analysis of seismic hazard probabilities. However, **Mase (2020)** recommends considering past seismic events as a reference in structural design for high-seismic areas. Therefore, a comprehensive analysis of potential seismic damage should be conducted to accurately assess the building's performance. Given the area's susceptibility to earthquakes, it is crucial to prioritise the seismic resistance of buildings. An initial assessment of the building's performance is essential to gauge its seismic resilience. Careful consideration of the seismic source and its impact on the building is essential. The careful selection of seismic ground motions, ground response analysis diagrams, and structural dynamic analysis are crucial in evaluating the building's performance and determining the reliability of the seismic design code. These numerical studies serve as a preliminary investigation before experimental tests are conducted.

This study evaluates the Integrated Laboratory Building of the Political and Social Science Faculty at the University of Bengkulu, constructed in a swampy area. The research begins with a soil investigation to gather data on the soil layers. Geotechnical investigation and geophysical measurements are conducted to understand the geological conditions in the study area. The ground motion from the Bengkulu-Mentawai Earthquake is used to analyse the seismic response of the building, considering the motion transmitted from the engineering bedrock through the soil layers. The ground motion at the depth where the pile tip is installed is employed for the dynamic analysis of the building. The study presents information on internal forces and the stress ratio. Additionally, it discusses the implementation of retrofitting methods to address damage caused by earthquake forces. This study offers insight into the integrated approach for assessing a building's performance.

2. Material and Method

2.1. Seismotectonic Settings and Geological Condition

Bengkulu, situated on the west coast of Sumatra Island near the Bukit Barisan Mountains and the Indian Ocean, faces significant tectonic challenges. The region is characterised by active tectonic sources, with the Sumatra Subduction Zone being the primary cause of powerful earthquakes, including the devastating Aceh Earthquake in 2004 and the Bengkulu-Mentawai Earthquake in 2007. Notably, Bengkulu city experienced the destructive Bengkulu-Mentawai Earthquake in 2007 and the Bengkulu-Enggano Earthquake in 2000, both attributed to this subduction activity (**Mase, 2022**). The Bengkulu-Enggano Earthquake in 2000, with a magnitude exceeding M_w 7.0, caused severe structural and infrastructural damage, emphasising the critical need for meticulous consideration in structural and foundation designs. Reports by **Hausler and Anderson (2007)** revealed extensive structural damage and the occurrence of liquefaction along the coastal area of Bengkulu city. Furthermore, **Mase et al. (2019, 2022b)** and **Sukkarak et al. (2021)** highlighted that liquefaction can result from reduced effective stress due to excessive pore water pressure. These seismic events underscore the urgency of careful consideration in structural and foundation designs to mitigate potential seismic damage.

Bengkulu is at risk of potential earthquakes due to several active faults, including the Sumatra and Mentawai faults. These strike-slip faults can cause low to moderate earthquakes with shallow focal depths. The Mentawai fault, located between Sumatra Island and the Sumatra Subduction, has caused several earthquakes in the western part of Sumatra Island, affecting Bengkulu city. Notable earthquakes triggered by this fault include the 2009 Padang Earthquake (**McCloskey et al., 2010**) and

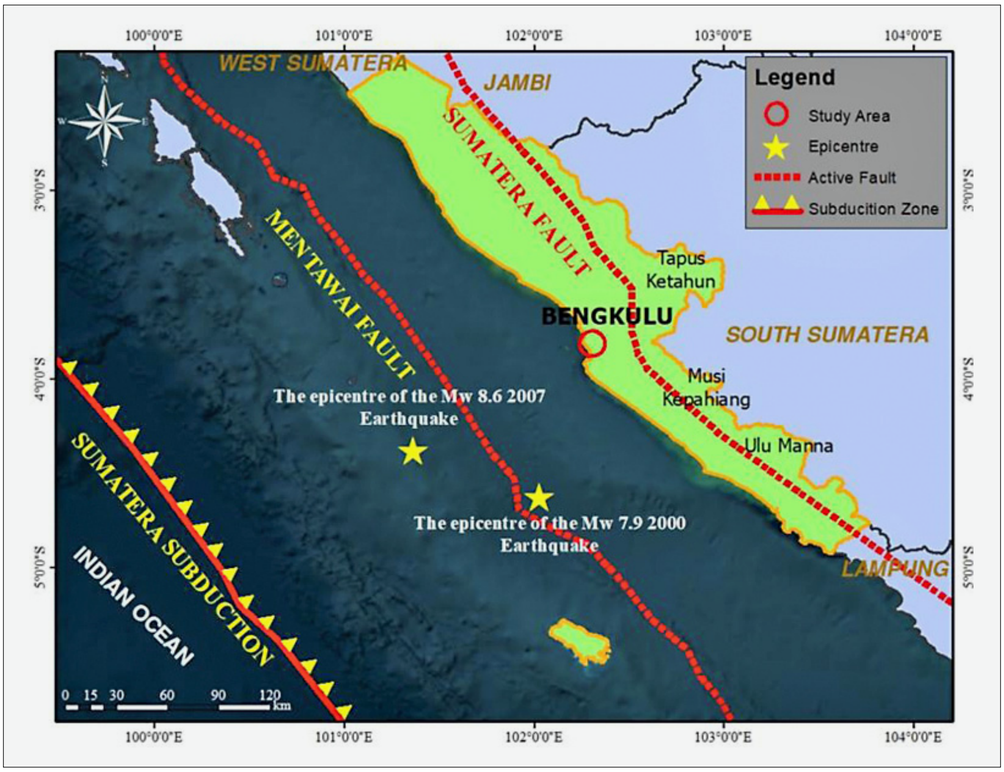

Figure 1: The seismotectonic setting of Bengkulu Province (redrawn based on **Mase et al., 2021a**)

the 2010 Mentawai Earthquake (**Prasidya et al., 2021**). On the mainland of Sumatra Island, the Sumatra fault, comprised of the Ketahun, Kepahiang, and Ulu Manna segments, poses a potential earthquake threat to Bengkulu city despite its distance. Historical records, such as the devastating earthquake on the Musi fault segment on December 15, 1979, reported by **Hadi et al. (2018)**, underscore the potential for significant seismic damage in the region of Kepahiang Regency.

Bengkulu city, located in the Bengkulu Basin, features diverse geological formations, including the Bintunan Formation (QTb), alluvium (Qa), reef limestone (Ql), swamp deposits (Qs), alluvium steps (Qat), and andesite (Tpan) (**Mase, 2022**). The predominant formations in Bengkulu city are the alluvium steps (Qat). According to **Mase et al. (2021a)**, Bengkulu city is classified into two main site classes: Site Class C and Site Class D. Site Class C is typically found in the eastern part of Bengkulu city, while Site Class D is generally located in the western part of Bengkulu city.

2.2. *Site Investigation Data and Earthquake Motion*

The layout in **Figure 2** shows the study area, which is the Integrated Laboratory Building of the Political and Social Science Faculty at the University of Bengkulu in the Muara Bangkahulu District. Muara Bangkahulu is the name of the main river in Bengkulu city and is part of Muara Bangkahulu's sub-watershed (**Mase et al., 2022c**). **Farid and Mase (2020)** pointed out swamp deposit areas dominated by peat land in several locations in the northern part of Bengkulu city, including the site of the investigated building. Additionally, **Mase et al. (2021a)** mentioned that the Muara Bangkahulu sub-district is located in an area with rock formations composed of alluvium traces (Qat), which falls under Site Class D. **Hollender et al. (2018)** also highlighted the increased susceptibility to seismic damage in areas categorised as Site Class D, which is in line with the study area. The report by **Hausler and Anderson (2007)** emphasised the common occurrence of seismic damage in the Muara Bangkahulu District during the 2007 Bengkulu-Mentawai Earthquake. Areas dominated by Site Class D are more vulnerable to significant impact due to lower soil resistance, as confirmed by a study conducted by **Farid and Mase (2020)**, reporting a relatively high seismic vulnerability index in Muara Bangkahulu Districts (**Mase et al., 2024**).

The findings of the site investigation in the study area are presented in **Figure 3**. This includes the soil profile, corrected standard penetration test values $(N_1)_{60}$, and shear wave velocity (V_s) profile. The figure shows the presence of three significant soil layers. The first layer is a 2.2 m thick clay layer with an average $(N_1)_{60}$ value of 4 blows/ft. This is followed by a 10.4 m thick first sand layer with an average $(N_1)_{60}$ of 12 blows/ft, then a final sand layer approximately 23.6 m thick with an average $(N_1)_{60}$ value of 50 blows/ft. The data emphasises the importance of the stiff layer located at a depth of 16 m, based on the $(N_1)_{60}$ value.

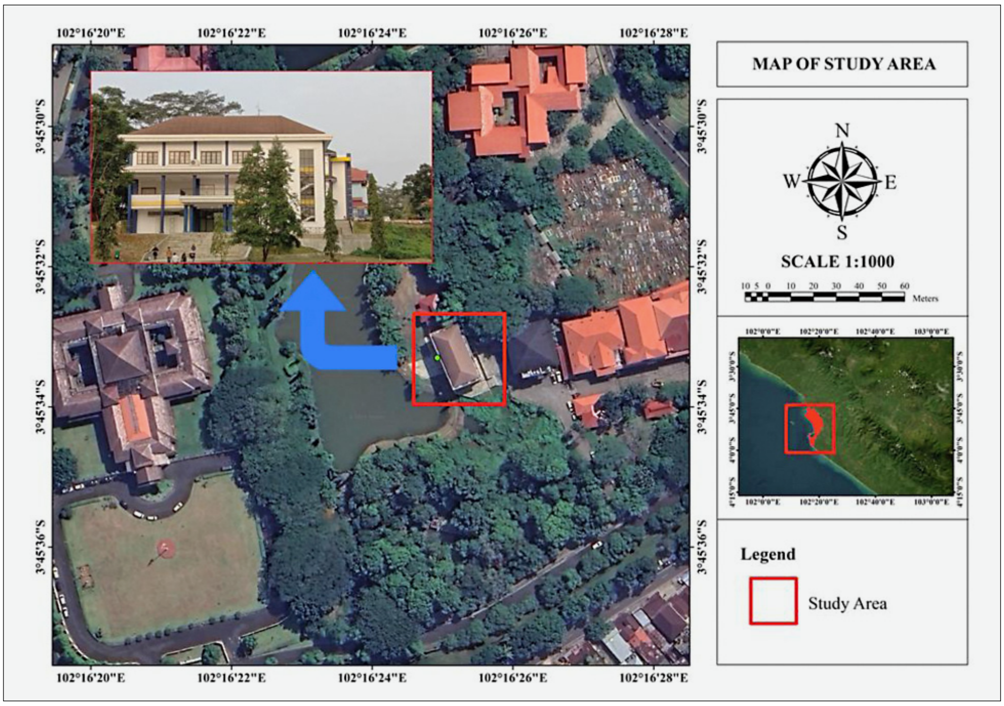

Figure 2: The layout of the study area

According to **Viani (2021)**, the engineering bedrock depth at the University of Bengkulu is around 33.9 m below the ground surface. Additionally, **Mase et al. (2024)** suggested that in the flood plain of Muara Bangkahulu River, including the University of Bengkulu, the engineering bedrock is generally found at depths ranging from 5 to 99 m beneath the ground surface. Both **Viani (2021)** and **Mase et al. (2024)** indicate a similar tendency for the engineering bedrock in the study area. **Miller et al. (1999**) state that engineering bedrock can be identified for surfaces with V_s over 760 m/s. Therefore, the 33.9 m depth could be assumed as the position of the engineering bedrock surface, as the V_s value is 796 m/s. According to the **Building Seismic Safety Council or BSSC (2020)**, the investigated site falls under Site Class D. This classification is based on the time-averaged V_s for the first 30 m depth (V_{s30}) being approximately 296 m/s, which falls within the range of 180 m/s to 360 m/s.

2.3. *Reinforced Concrete of the investigated building*

The Integrated Laboratory Building of the Faculty of Political and Social Sciences comprises three floors with several laboratories and a meeting room: a social welfare Laboratory, a Public Administration Laboratory, a Sociology Laboratory, a Communication Science and Journalism Multimedia Laboratory, and an International Meeting Room. The building was constructed using reinforced concrete based on the guidelines of **SNI 1726:2019 (2019)**. **Figure 4** illustrates the elements and dimensions of the building structure, including connecting beams, columns, floor plates, pile caps, and bore piles. **Table 1** provides information on the dimensions of the structural elements. Generally, the building is approximately 12 meters in height, with a width of 11 meters and a length of 24 meters. The reinforced concrete's compressive strength (*fc¢*) specification is 21 MPa.

2.4. *Research Framework*

As depicted in **Figure 5**, the analytical framework commenced with a meticulous collection of site investigation data. This encompassed obtaining boring logs, soil profiles, and soil resistance data through an extensive field survey involving drilling and sampling at various locations within the study area. In addition, comprehensive data on building information and ground motion during the Bengkulu-Mentawai Earthquake 2007 were diligently gathered from reliable databases and cross-referenced with local records to ensure accuracy. Notably, the seismic ground motion data from the Bengkulu-Mentawai Earthquake, based on **Mase (2020)**, is widely recognised as the most credible earthquake in Bengkulu city and forms a crucial component of this assessment. **Tanapalungkorn et al. (2020)** have also emphasised the importance of carefully selecting seismic ground motion for hazard analysis, emphasising the essence of representative ground motion before conducting seismic ground response analysis. Finally, the culmination of this process was the seismic ground response analysis, the main focus of our study.

The input motion for ground response analysis used in this study is based on the ground motion utilised by **Mase (2017)** for seismic ground response analysis in Bengkulu

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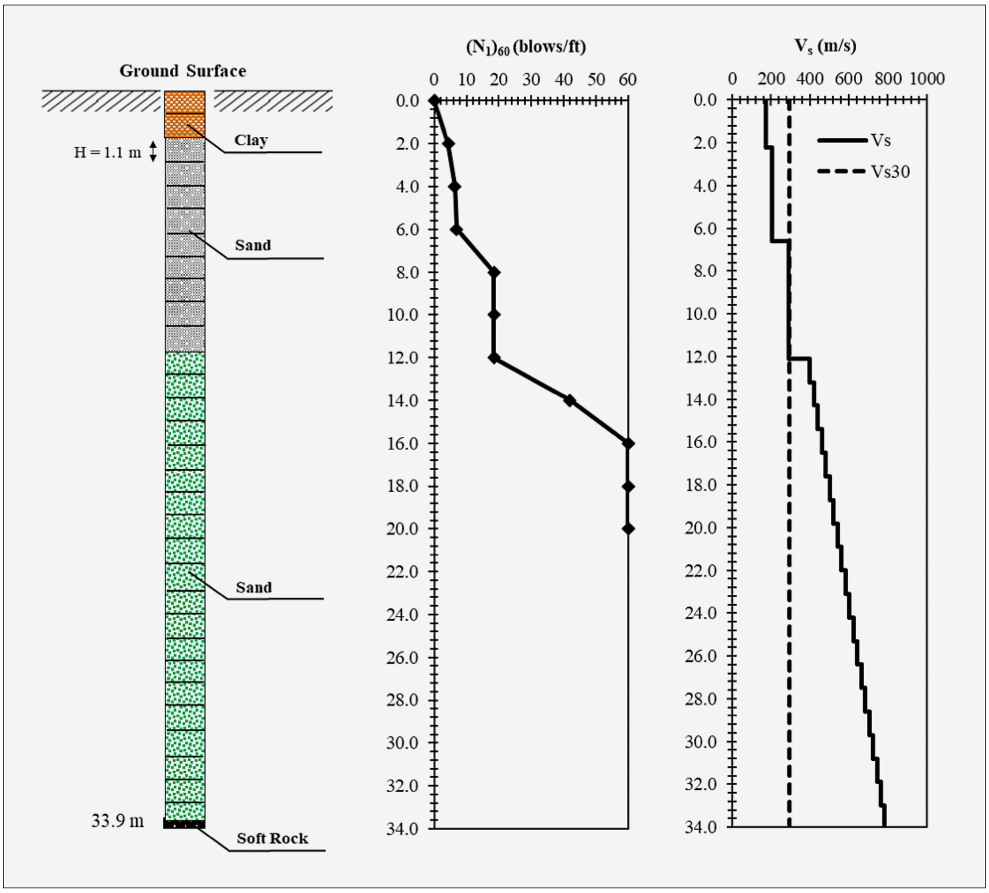

Figure 3: Condition of soil layers from site investigation

city. Due to the data limitation, the ground motion used in this study is the maximum horizontal ground motion (in the east-west (ew) direction) of the Bengkulu-Mentawai Earthquake, which is predicted to occur in the study area. The peak ground acceleration (PGA), peak ground velocity (PGV), and peak ground displacement (PGD) are 0.154g, 0.124 m/s, and 0.097 m, respectively, as shown in **Figure 6**. All three components of ground motion should be used for a better analysis.

The study utilises the pressure-dependent hyperbolic (PDH) model proposed by **Hashash et al. (2020)**, known for its precision and accuracy. The next step involves conducting a nonlinear one-dimensional seismic response analysis using this model to address the vertical propagation of horizontal shear waves through soil layers (**Hashash and Park, 2001**). The nonlinear analysis provides more accurate and realistic ground movement modelling results consistent with field evidence (**Puri et al., 2018**). **Mase et al. (2023)** also noted that nonlinear one-dimensional ground response analysis could observe soil behaviour for specific purposes, such as liquefaction during earthquakes. The initial stage of nonlinear analysis involves modelling soil layers based on physical and dynamic parameters, including shear wave velocity (V_s), layer thickness (h), bulk density (γ), plasticity index (PI), and pressure wave velocity (V_p), as shown in **Figure 3**.

The element size (d) is not just a factor but an essential consideration in conducting nonlinear analysis. Its impact on result accuracy must be balanced. The element size, often determined through correlation with the maximum frequency (f_{max}), as stated by **Hashash et al. (2020)**, is a parameter that demands attention. Therefore, **Mase et al. (2022b)** have proposed the wavelength analysis method, a significant step towards minimising the high-frequency effect in seismic response analysis, as expressed in the following equation,

$$d = \frac{V_s}{4 f_{\max}} \tag{1}$$

In **Equation 1**, d represents element thickness, V_s is the minimum shear wave velocity for the calculated layer, and $f_{\max}$ is the maximum frequency of 33 Hz. After using **Equation 1** for calculation, the element thickness of each layer considered in this study is 1.1 m. The plasticity index (PI), as proposed by **Fener et al. (2005)**, can be obtained by correlating with V_p. According to **Mase et al. (2021a)**, the correlation between V_p and V_s can be used to determine the V_p value. For clayey soil, PI is estimated based on the following equations,

$$V_p = 1.25 V_s \tag{2}$$

$$PI = -16.49 \ln \ln(V_p) + 121.95 \tag{3}$$

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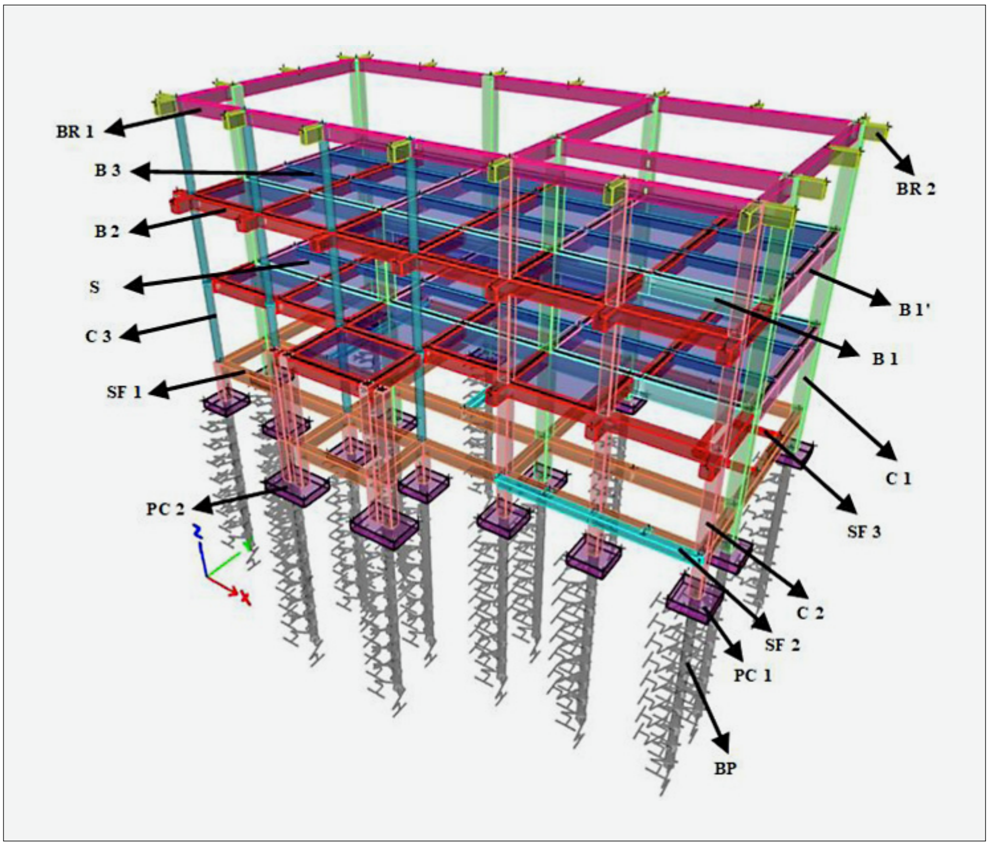

Figure 4: 3D modelling of Integrated Laboratory Building of Political and Social Science Faculty structures

In **Equations 2** and **3**, V_p and V_s denote pressure wave and shear wave velocities, respectively.

Table 2 presents the physical parameters of the soil utilised for soil modelling. The dynamic parameters of damping ratio (ξ) and shear modulus ratio (G/G_{max}) are confidently derived from well-established reference curves. The **Vucetic and Dobry (1991)** curve is utilised for cohesive soil types, considering the plasticity index (PI) value. The **Seed and Idriss (1970)** curve is applied for granular soil types under mean limit conditions and does not consider the PI value. **Hashash et al. (2020)** proposed several curve-fitting parameters, and the soil model's parameters (*b*, *s*, *b*, and *d*) are generated based on recommendations from **Hashash et al. (2020)**.

Various parameters are collected to observe the ground response during the earthquake, such as the time history of acceleration and spectral acceleration. The analysis involves applying motion to the engineering bedrock surface and observing how the wave propagates through the layer. In this study, the motion at the layer where the pile tip of the building is installed is used for dynamic analysis. Notably, the pile tip is placed 10 meters below the ground surface, a critical factor in the methodology. The results of this analysis include peak ground acceleration (PGA) values and acceleration spectra (SA). Additionally, the study discusses the amplification factor (AF) of ground motion.

The final step involves analysing the structural performance using FEM. The study includes FEM for dynamic analysis of time history. The analysis begins with creating geometric modelling of the structural elements of the Integrated Laboratory Building of the Political and Social Science Faculty. Structural loading refers to the Indonesian seismic design code **SNI 1727:2013 (2013)**. The loading involves inputting loads in load patterns, load cases, and load combinations. The earthquake wave is an accelerogram resulting from the one-dimensional seismic response analysis in the previous step. Modelling assumptions for the soil layers in the foundation are made by entering spring values. The spring value is determined by establishing the permitted bearing capacity of the bored pile foundation using the **Aoki and De Lancer (1975)** method. The bearing capacity is then used to determine the spring constant value in the vertical (K_{sv}) and horizontal (K_{sh}) directions, as suggested by **Bowles (1997)**. The spring constant values for each layer can be found in **Table 3**. The analysis results provide structural responses in normal force, shear force, and bending moment. Unsafe structural elements or overstrength elements are marked in red. These unsafe elements are then analysed for repair methods using retrofitting by concrete jacketing. **Habib et al. (2020)** and **Hong et al. (2021)** suggest that this retrofitting solution enlarges the cross-section of the structure and adds reinforcement and strip bars to strengthen weak structural

Table 1: Structure element data

Elements of Structure	Notation	Dimension (mm)	Thickness (mm)	Characteristic of concrete compression (f'_c) (MPa)	Span Length of Beam (m)	Storey height (m
Connecting Beam 1	SF 1	250 × 500	-	21	-	-
Connecting Beam 2	SF 2	150 × 300	-	21	-	-
Connecting Beam 3	SF 3	150 × 200	-	21	-	-
Beam 1	B 1	350 × 700	-	21	8	-
Beam 1′	B 1′	350 × 700	-	21	8	-
Beam 3	B 3	250 × 500	-	21	8	-
Beam 4	B 4	250 × 400	-	21	8	-
Beam Ring 1	BR 1	250 × 500	-	21	-	-
Beam Ring 2	BR 2	250 × 500	-	21	-	-
Column 1	C 1	400 × 500	-	21	-	4
Column 2	C 2	400 × 400	-	21	-	4
Column 3	C 3	400	-	21	-	4
Slab	S	-	125	21	-	-
Pile Cap 1	PC 1	1500 × 1500	-	21	-	-
Pile Cap 2	PC 2	1500 × 2200	-	21	-	-
Bore Pile	BP	400	-	21	-	-

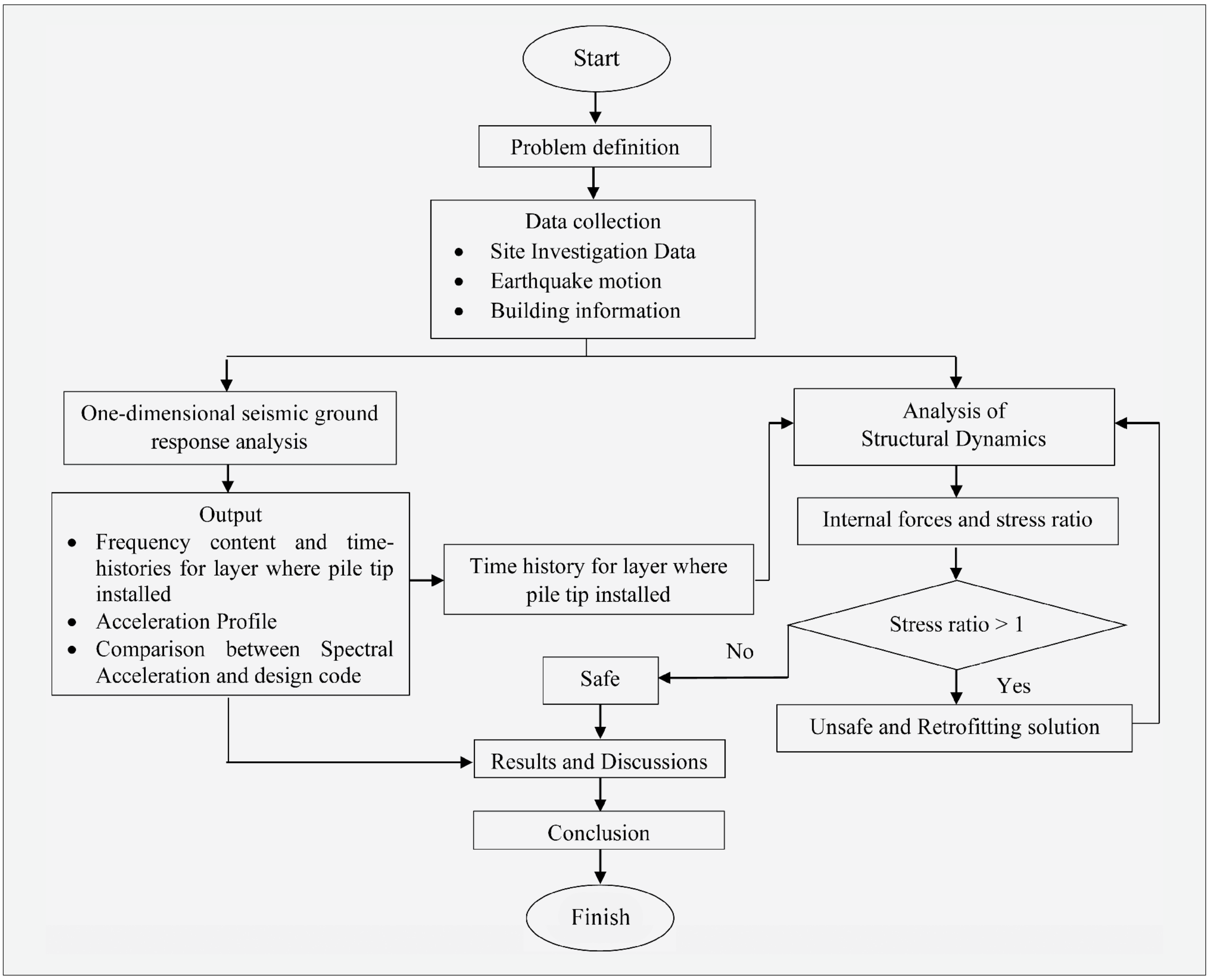

Figure 5: Research framework

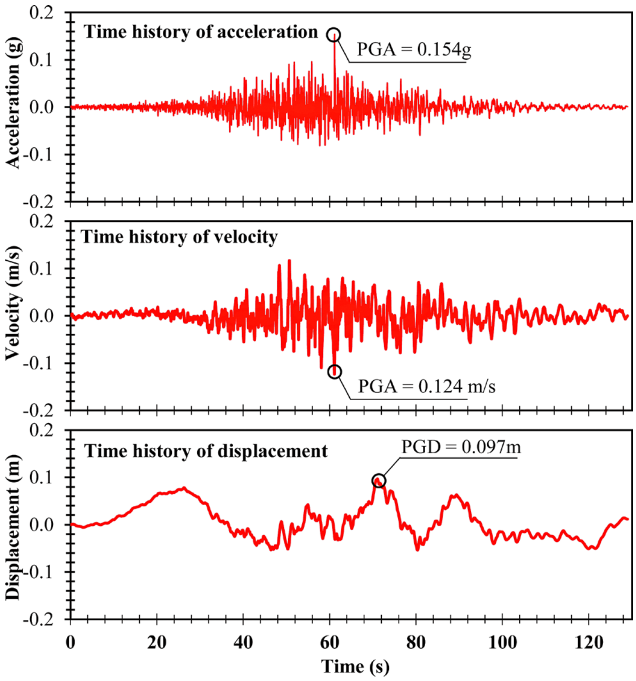

Figure 6: The M_w 8.6 Bengkulu-Mentawai Earthquake in 2007 ground motion time histories (modified from Mase, 2017)

elements. Furthermore, dynamic analysis checks the building's performance after retrofitting treatment.

3. Results

3.1. Site response

Seismic wave propagation analysis using a PDH model provides several ground motion parameters, such as PGA, AF, and SA. The highest ground acceleration value or PGA is crucial in assessing earthquake impact over a specific period. In **Figure 7**, the PGA value at layer 1 is 0.220g, the input motion is 0.154g, and layer 9 is 0.147g. Comparing the output and input PGA values gives AF. AF, which indicates changes in earthquake acceleration values from the bedrock to the surface, is 1.429 in the surface layer and 0.955 in the foundation layer. The presence of clay soil can significantly amplify the factor at the ground surface, acting as a weak layer that influences the elastic response during earthquake shaking. Bedrock with high stiffness produces a significant AF (**Misliniyati et al., 2019; Mase et al., 2024**). This is due to the differences in V_s for each soil layer. A smaller V_s value indicates a greater AF value. Therefore, a near-surface layer with a small V_s value tends to have a significant AF (**Finn and Ruz, 2016**). These findings have practical implications for understanding and predicting earthquake response, particularly in areas with clayey soil or high-stiffness bedrock.

The comprehensive analysis results, featured in **Figure 8**, showcase the comparisons of spectral acceleration. Notably, the designed spectral acceleration values

Table 2: Input parameters for soil modelling

Layer	*h* (m)	γ (kN/m³)	V_s (m/s)	V_p (m/s)	*PI*
Clay	1.1	19.04	172.93	216.16	33
Clay	1.1	18.27	172.93	216.16	33
Sand	1.1	18.53	205.42	256.78	-
Sand	1.1	18.30	205.42	256.78	-
Sand	1.1	18.12	205.42	256.78	-
Sand	1.1	17.98	205.42	256.78	-
Sand	1.1	19.13	291.08	363.85	-
Sand	1.1	19.03	291.08	363.85	-
Sand	1.1	18.94	291.08	363.85	-
Sand	1.1	18.86	291.08	363.85	-
Sand	1.1	18.79	291.08	363.85	-
Sand	1.1	19.87	399.93	488.69	-
Sand	1.1	20.00	420.26	514.70	-
Sand	1.1	20.11	440.59	540.71	-
Sand	1.1	20.22	460.92	566.71	-
Sand	1.1	20.33	481.25	592.72	-
Sand	1.1	20.44	501.58	618.73	-
Sand	1.1	20.54	521.91	644.74	-
Sand	1.1	20.64	542.24	670.75	-
Sand	1.1	20.74	562.57	696.76	-
Sand	1.1	20.83	582.90	722.77	-
Sand	1.1	20.92	603.23	748.78	-
Sand	1.1	21.01	623.56	774.79	-
Sand	1.1	21.10	643.89	800.80	-
Sand	1.1	21.18	664.22	826.81	-
Sand	1.1	21.26	684.55	852.82	-
Sand	1.1	21.34	704.88	878.83	-
Sand	1.1	21.42	725.21	904.83	-
Sand	1.1	21.49	745.54	930.84	-
Sand	1.1	21.56	765.87	956.85	-
Sand	0.9	21.62	782.51	978.13	-

specified in **SNI 1726:2012 (2012)** and **SNI 1726:2019 (2019)** for Bengkulu city consistently indicate lower values than those obtained from the ground response analysis for the foundation layer. These values are essential in seismic design as they signify the maximum expected ground acceleration at a specific location. Furthermore, the one-dimensional nonlinear seismic response analysis also revealed varying spectral acceleration results for each period, as depicted in **Figure 8**. It is evident from the analysis findings that the application of the input waveform leads to a gradual increase in spectral acceleration within the 0.2 second to 0.4 second period. Noteworthy peaks include a maximum spectral acceleration of 0.802g at 0.253 seconds on the ground surface and 0.465g at 0.345 seconds on the foundation base. This increase in spectral values on the surface and foundation layer is attributed to the influence of soil density, with denser soil indicating smaller ground acceleration values.

Table 3: Soil layer modelling spring constant value

Soil Layers	F (kN/m²)	Q_s (kN/m²)	Q_u (kN/m²)	Q_a (kN/m²)	K_{sv} (kN/m³)	K_{sh} (kN/m³)
Clay (Layer 1)	2.338	26.437	294.973	117.989	14158.692	28317.385
Clay (Layer 2)	6.448	72.920	341.456	136.582	16389.870	32779.741
Sand (Layer 3)	7.938	282.542	551.078	220.431	26451.721	52903.441
Sand (Layer 4)	12.758	144.293	412.829	165.131	19815.773	39631.546

Table 4: Mode shape and mass participation ratio of structure

Mode Shape	T (seconds)	UX	UY	RZ	SumUX	SumUY	SumRZ	Information
1	0.788	0.672	0.003	0.209	0.672	0.003	0.209	Translation X
2	0.736	0.005	0.843	0.000	0.676	0.846	0.209	Translation Y
3	0.611	0.189	0.001	0.624	0.865	0.847	0.833	Rotation Z
...	...	...	...	...	...	...	...	...
130	0.032	0	0	0	1	1	1	-

3.2. Dynamic response of the building

According to **SNI 1726:2019 (2019)**, the natural vibration period of a structure should not exceed the product of the coefficients for the upper limit on the calculated period. This requirement is intended to ensure resident comfort and minimise damage to the building structure. Following the seismic design code's calculation procedure, the Integrated Laboratory Building of the Political and Social Science Faculty has a structural vibration period limit of 0.610 seconds. However, analysis using FEM reveals a structural vibration period of 0.788 seconds, as shown in **Table 4**. This indicates that the building's vibration period exceeds the allowable limit, which could have unfavourable consequences for the building.

Mode shape refers to a building structure's vibration pattern when subjected to earthquake loads. **SNI 1726:2019 (2019)** sets regulations for regular buildings, specifying that the first and second vibrations should be dominant in the translational direction, with the third vibration being rotational. It is crucial to include an adequate number of mode shapes in structural dynamics analysis to capture the significant dynamic behaviour of the structure. The mass participation factor indicates the percentage of total mass participating in each mode. The sum of the mass participation factors of all modes should approach 100% to ensure an accurate representation of the structure's dynamic characteristics. **Table 4** also presents the analysis of mass participation variations in the integrated Laboratory Building of Political and Social Science Faculty, obtained for the first and second variations of the building moving in the translational direction and for the third vibration variation moving in the rotational direction. Mass participation reaches 100% when the number of mode shapes reaches 130, indicating that all earthquake forces have been thoroughly analysed. This condition is based on the criteria outlined in **SNI 1726:2019 (2019)**.

This study examined the internal force values before and after the application of the load. The interconnected floor system of the Integrated Laboratory Building of the Political and Social Science Faculty produces internal forces from the combination of dead, live, and earthquake loads. Through the use of FEM analysis, the accurate representation of the distribution of internal forces after exposure to earthquake loads is depicted in **Figure 9**. **Figure 9a** showcases the distribution of the bending moment diagram (BMD), **Figure 9b** illustrates the distribution of the shear force diagram (SFD), and **Figure 9c** presents the distribution of the normal force diagram (AFD).

The detailed data in **Table 5** presents the maximum internal forces acting on the structure before and after earthquake loads. Before the earthquake load is applied, the structure exhibited a range of maximum normal force from 0.358 kN to 834.280 kN, maximum shear force from 4.031 kN to 134.692 kN, and maximum bending moment from 2.062 kNm to 164.054 kNm. Following the earthquake loads, the structural analysis revealed an insightful summary of the maximum internal forces acting on each structural element. The maximum normal force now ranges from 26.326 kN to 1927.873 kN, with an average percentage increase of 78.676%. The maximum shear force ranges from 4.214 kN to 331.703 kN, with an average percentage increase of 61.866%. The maximum bending moment ranges from 2.227 kNm to 384.767 kNm, with an average increase percentage of 57.055%.

The structural reinforcement inspection is crucial for evaluating the stress ratio of elements. This inspection is essential to ensure that each element meets the allowable criteria. The stress ratio, which is the ratio of the maximum internal force acting on a structure to the allowable strength of each element, plays a vital role in determining the structure's safety (**Mase et al., 2022a**). If the stress ratio value exceeds one, it is imperative to consider dimensional adjustments. In cases where this occurs in an existing building during the design stage, structural strengthening action is necessary. **Figure 10** depicts the stress ratio of the concrete examined in this study, highlighting two beam elements (indicated by the red line) with a stress ratio exceeding one, indicating over-strength. The over-strength of these beam elements

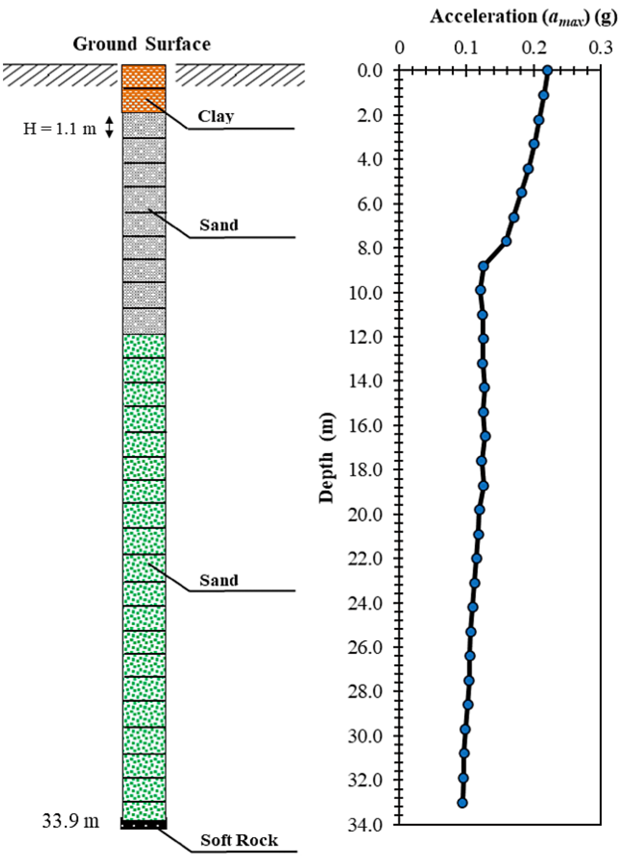

Figure 7: Profile of acceleration based on seismic ground response analysis

is attributed to the horizontal force resulting from earthquake shaking.

Excessive shear and torque forces could trigger structural weakening (**Alam et al., 2024**). Stirrup reinforcement is crucial for the weakened element due to excess shear force and torque (**Altunsu et al., 2024**). Beam element B2 is currently experiencing weakening due to external forces from earthquake shaking. As the beam is designed to withstand horizontal loads, it is essential to enhance its performance. While the column element remains generally safe, the stress ratio value falls within the critical zone (magenta colour), indicating potential seismic damage. Although it meets the requirements for earthquake-resistant buildings (**Yu et al., 2020**), damage is possible, especially during significant shaking. This situation underscores the need for the structure to maintain its strength, allowing people in the building to escape during a large earthquake.

3.3. Concern about the building for future development

The structural performance analysis results are based on earthquake-resistant building criteria, with the weakened beams being a key concern. It is crucial to consider this when repairing weakened elements. Considering the building's function as a public space, it is imperative to implement a retrofitting repair method, such as concrete jacketing, for the two weakened beam structural elements.

Figures 12a, b, and **c** illustrate the cross-section of beam B2, which has weakened. An analysis was then conducted using various methods. The beam is identified as a weak structural element, so it can be repaired using the concrete jacketing method. The design for reinforcing the beam using the concrete jacketing method involves enhancing the structural dimensions. The beam to be strengthened is supplemented with flexible reinforce-

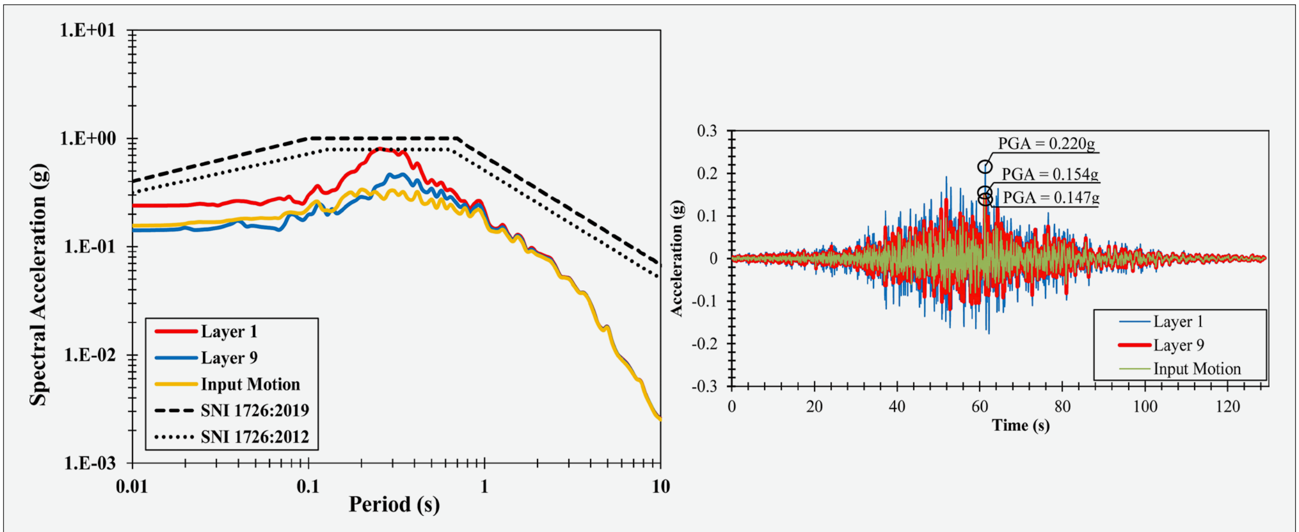

Figure 8: Comparison of spectral acceleration and time history of acceleration

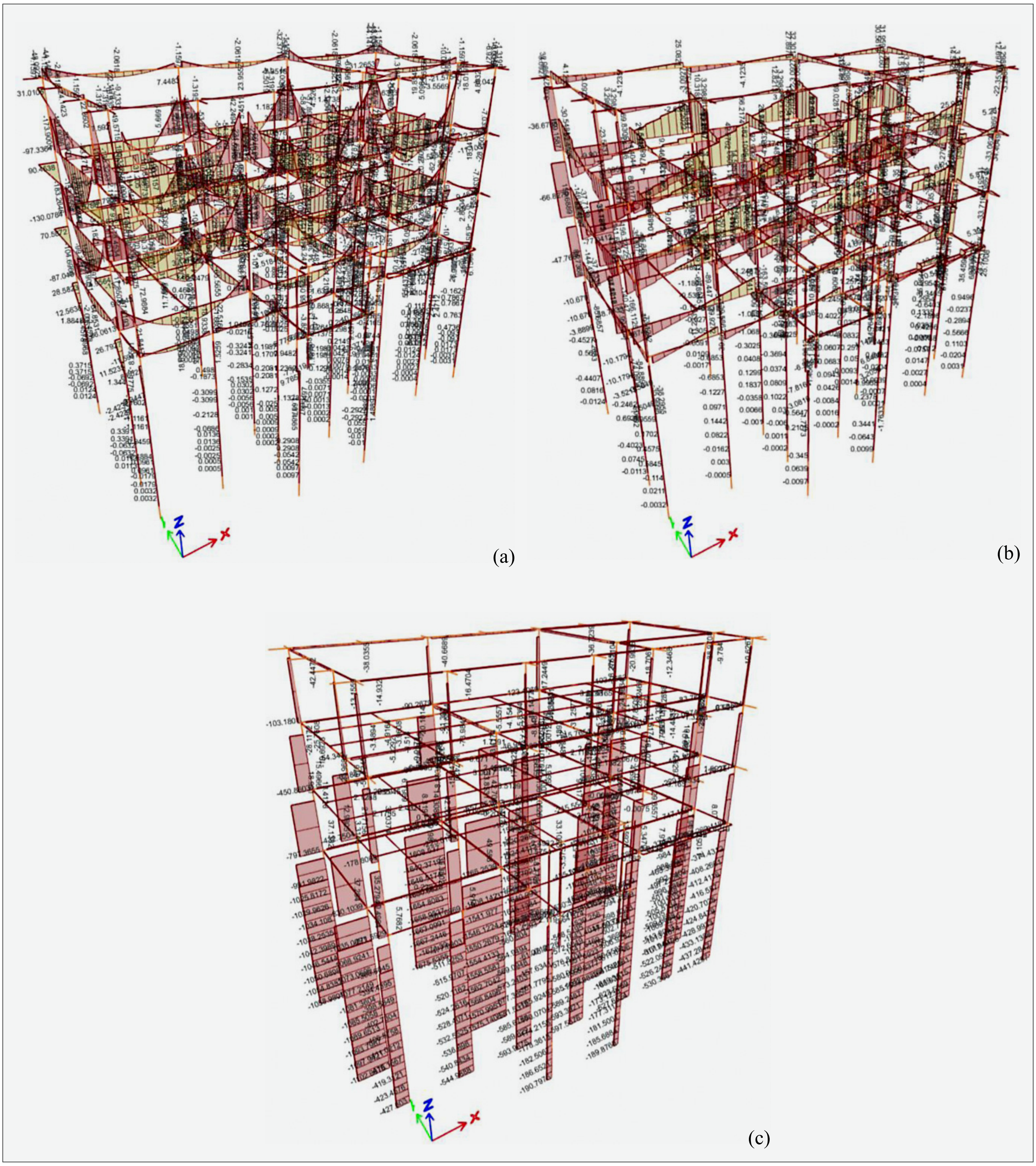

Figure 9: Internal force diagram after applying earthquake loads (a) BMD, (b) SFD, and (c) NFD

ments, shear reinforcements, and concrete blankets to support the load adequately. **Figures 12d**, **e**, and **f** depict the beam dimensions and the reinforcement improvement under the concrete jacketing method. The analysis results indicate that with a 53.12% increase in dimensions and 6D16 reinforcement (6 deformed steel reinforcements), Beam B2, which was previously weakened, has become safe.

After the concrete jacketing process, **Figure 13** depicts the internal forces acting on the structure. Addressing the weak beam has significantly reduced the potential for failure due to earthquake shaking compared to the previous distribution. Furthermore, **Figure 13** demonstrates the stress ratio distribution after the retrofitting countermeasure, revealing the absence of red zones. This indicates that the structure is now considerably safer than before. Thus,

Table 5: Comparison of maximum internal forces before and after an earthquake load is applied

Element	Maximum Internal Forces						Percentage Increase (%)		
	Before			After					
	Normal Force (kN)	Shear Force (kN)	Moment (kNm)	Normal Force (kN)	Shear Force (kN)	Moment (kNm)	Normal Force (kN)	Shear Force (kN)	Moment (kNm)
SF 1	31.610	31.884	37.388	197.147	89.447	96.933	83.967	64.354	61.429
SF 2	2.070	6.984	8.981	69.560	203.960	176.102	97.024	96.576	94.900
SF 3	0.358	4.031	6.418	52.691	13.705	18.989	99.321	70.586	66.204
B 1	32.639	97.524	139.975	301.100	244.013	333.965	89.160	60.033	58.087
B 1’	56.799	134.692	164.054	211.843	331.703	384.767	73.188	59.394	57.363
B 2	14.709	59.434	82.979	350.890	192.003	179.481	95.808	69.045	53.767
B 3	5.222	35.066	47.630	43.982	77.477	103.039	88.128	54.740	53.775
BR 1	35.177	23.072	33.909	81.315	52.089	68.230	56.740	55.705	50.302
BR 2	0.757	4.124	2.062	26.326	4.124	2.227	97.126	0.000	7.401
C 1	834.280	75.085	133.461	1927.873	199.467	335.045	56.725	62.357	60.166
C 2	278.530	14.110	61.661	608.478	156.516	166.792	54.225	90.985	63.031
C 3	242.093	15.352	31.147	511.882	37.090	74.572	52.705	58.610	58.233

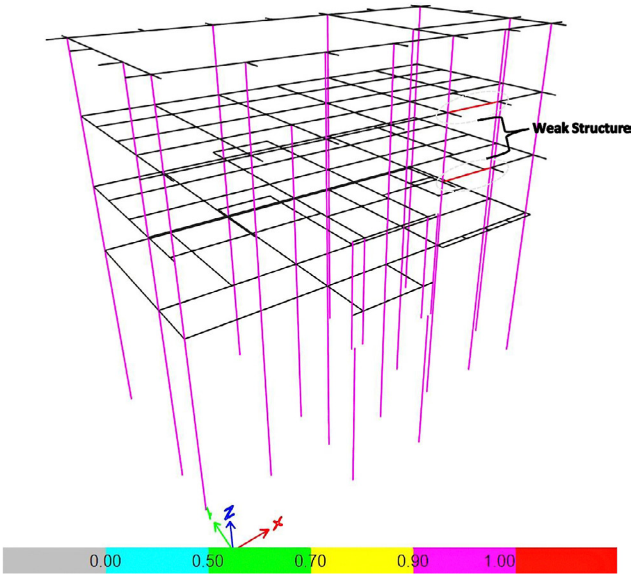

Figure 10: Stress ratio of concrete structure

implementing retrofitting methods to mitigate potential seismic damage in the future is crucial.

4. Conclusions

This paper presents numerical analyses to observe the performance of a monumental building at the University of Bengkulu, Indonesia. The study focuses on the Integrated Laboratory Building of the Political and Social Science Faculty. Geotechnical and geophysical investigations were conducted to obtain the site characteristics of the study area. One-dimensional seismic ground response analysis was performed to obtain soil response at the foundation. A structural dynamic analysis was also conducted to inspect the structure’s performance during earthquakes in the study area. The research draws several concluding remarks on the performance of the building, highlighting the application of one-dimensional wave analysis and structural analysis using FEM, as follows.

1. The seismic ground response analysis reveals the information: the peak ground acceleration (PGA) at the ground surface registers at 0.220g, with an amplification factor of 1.429, indicating potential seismic wave acceleration. The maximum spectral acceleration at the ground surface is recorded at 0.802g, resonating at 0.253 seconds. Furthermore, for the foundation layer, the measured PGA is 0.147g, with an amplification factor of 0.955 and a maximum spectral acceleration of 0.465g at 0.345 seconds.
2. The analysis using FEM reveals that the building period is beyond its natural limit. This presents a severe risk of structural failure, especially in the beams influenced by earthquake loads. The lack of confining pressure exacerbates this vulnerability. Immediate countermeasures are needed to ensure the safety and stability of the building.
3. The analysis of structural strengthening using the retrofitting method demonstrates that the weakened elements are now secure. With a 53.12% increase in dimensions, beam element B2 effectively resists shear forces on the beam at the edges.
4. This study presents the implementation of seismic response analysis and structural analysis for building assessment. The framework of this study can be applied to evaluate building performance in different regions. However, the reliability of the retrofit solution in response to updated seismic design codes still needs to be improved. Therefore, it is

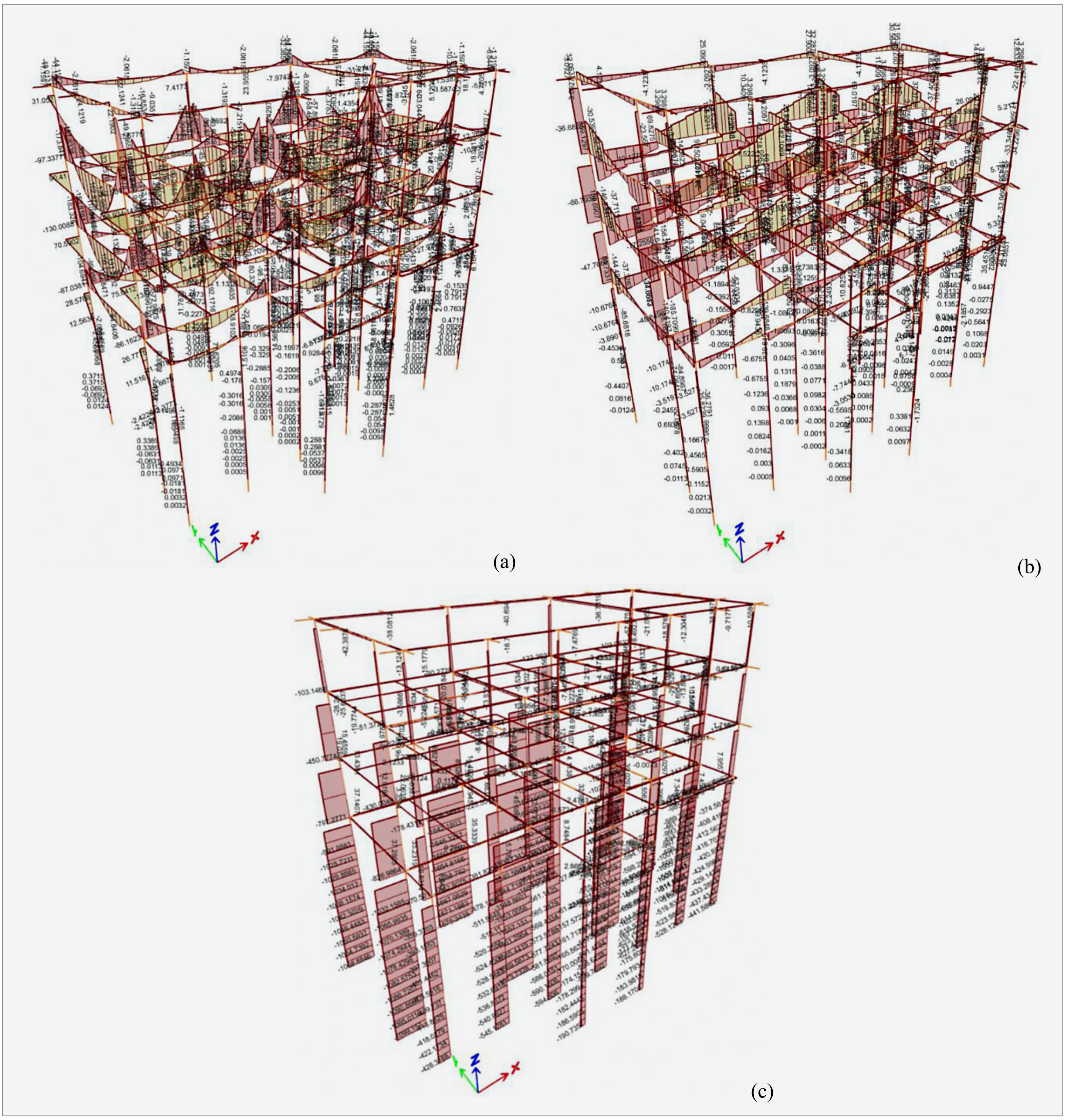

Figure 11: Internal force diagram after concrete jacketing (a) BMD, (b) SFD, and (c) NFD

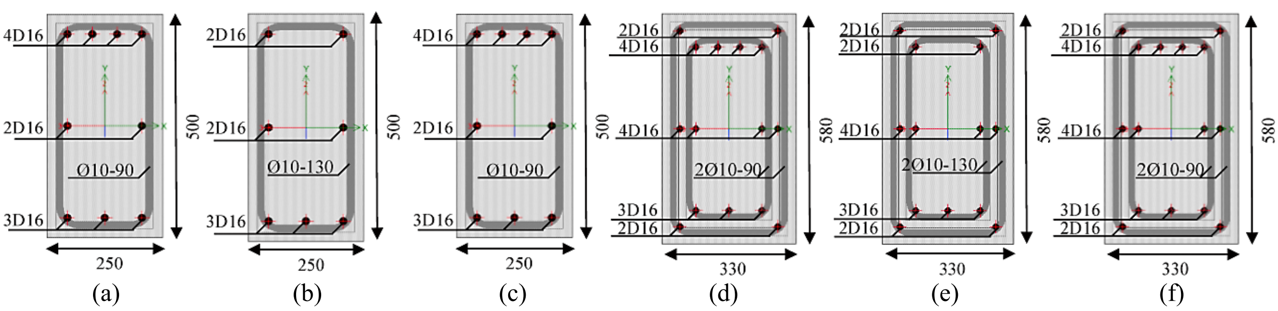

Figure 12: Dimension of B2 before and after retrofit (a) before concrete jacketing for left joint reinforcement (b) before concrete jacketing for field reinforcement (c) before concrete jacketing for proper joint reinforcement (d) after concrete jacketing for left joint reinforcement (e) after concrete jacketing for field reinforcement (f) after concrete jacketing for proper joint reinforcement

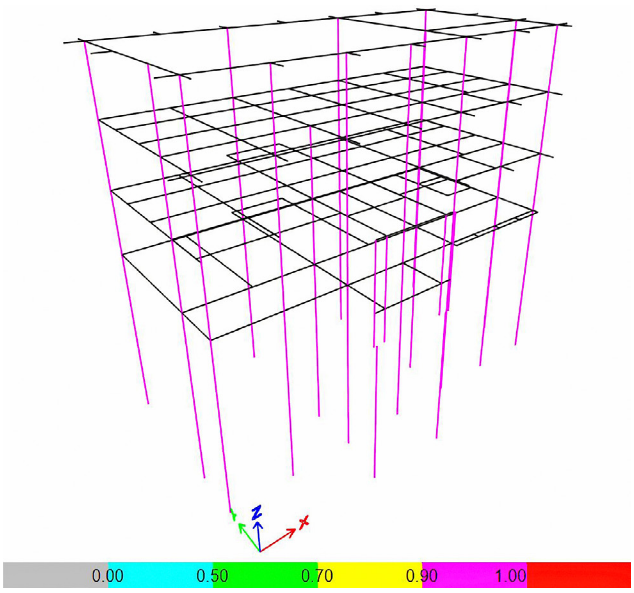

Figure 13: Stress ratio of concrete structure after retrofitting

crucial to conduct experimental tests to verify it. This critical step will be addressed in future studies.

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SAŽETAK

Numeričke analize za promatranje izvedbe monumentalne zgrade na Sveučilištu u Bengkuluu, Indonezija

Integrirana laboratorijska zgrada Fakulteta političkih i društvenih znanosti, smještena u području sklonom seizmičkim aktivnostima, predmet je sveobuhvatnoga proučavanja pomoću analize seizmičkoga odgovora. Ovo je istraživanje dalo važne rezultate, prikupivši ključne sekundarne podatke kao što su slojevi tla, dubina temelja, struktura zgrade i informacije o valovima potresa. Analiza seizmičkoga odgovora, provedena upućivanjem na sintetički val potresa iz potresa Bengkulu – Mentawai 2007. godine, magnitude 8,6 Mw, pružila je sveobuhvatan pregled valova potresa na slojevima temeljnoga tla. Podatci, analizirani metodom konačnih elemenata za razumijevanje strukturnoga odziva, otkrili su da su vršno ubrzanje tla (PGA) i faktor pojačanja na površinskome sloju tla 0,220 g, odnosno 1,429. Najkritičniji PGA i faktori pojačavanja na temeljnome sloju tla iznose 0,147 g i 0,955. Strukturna analiza otkrila je vrijednosti unutarnjih sila i deformacije strukturnih elemenata. Navedeno zahtijeva naknadnu prilagodbu strukturnih elemenata kako bi se smanjile deformacije. Jedna praktična i vrlo učinkovita metoda naknadne ugradnje uključuje povećanje dimenzija grede za 53,12 %. Naknadnom ugradnjom utjecaj strukturnih deformacija može se svesti na najmanju moguću mjeru, čime se povećava otpornost zgrade u slučaju potresa jednake ili veće jačine. Ti nalazi naglašavaju važnost naših istraživanja i ističu ulogu inženjera, arhitekata i istraživača u osiguravanju sigurnosti i dugovječnosti konstrukcija u područjima sklonim seizmičkim aktivnostima.

Ključne riječi:
faktor pojačavanja, izvedba zgrade, PGA, seizmički odgovor, naknadna ugradnja

Author's contribution

Lindung Zalbuin Mase (1) (Ph.D., Associate Professor, an expert on Geotechnical and Structural Engineering) provided site-specific response analysis and structural analysis, composed the original draft and editing, and held funding acquisition, supervision, and project administration. **Salsabhila Isdianty (2)** (B.Eng, Civil Engineering) provided site-specific response and structural analyses and composed the original draft. **Khairul Amri (3)** (Doctor, Associate Professor, and an expert on Environmental Engineering) provided supervision and composed the original draft.