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RELIABILITY STUDY OF SPECTRAL ACCELERATION DESIGNS AGAINST EARTHQUAKES IN BENGKULU CITY, INDONESIA

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ABSTRACT

This paper presents a reliability study of spectral acceleration designs against earthquakes in Bengkulu City. Seismic Hazard Analysis of 1,968 events is performed to define the controlling earthquake event. Furthermore, the controlling earthquake is used as the scale factor to generate five input motions for one-dimensional seismic response analysis. The spectral accelerations resulting from the analysis are then compared to the updated spectral acceleration design. The results show that spectral acceleration designs are still able to cover the spectral acceleration of seismic response analysis. However, for the short period, the spectral acceleration of seismic response analysis exceeds the designed spectral acceleration. This is a matter of concern, since most of the building natural period in Bengkulu City is still categorized as short. In general, this study brings awareness to the design aspect considering earthquakes in Bengkulu City in order to reduce the possible impact on structures in the future.

Keywords: Earthquakes; Seismic hazard analysis; Seismic response analysis; Spectral acceleration

1. INTRODUCTION

It is known that earthquakes are a natural hazard which could trigger massive damage to a region such as Indonesia (Kanata et al., 2014; Sukanta et al., 2015). In 2000, a 7.90 M_w earthquake occurred in Bengkulu Province, Indonesia. It resulted in destructive damage, including collapsed buildings, fatalities, and injuries. The earthquake also triggered other catastrophic hazards, such as landslides and liquefactions in mountainous and coastal areas in Bengkulu. Seven years later, another big earthquake with a magnitude of 8.6 M_w hit the area (Mase, 2017a). This earthquake also resulted in structural damage and other geotechnical phenomena, such as ground failure and liquefaction. During both events, Bengkulu City suffered more serious impact than other cities and regencies in Bengkulu Province, due to the fact that the released energy of the earthquakes in 2000 and 2007 was very large (Mase, 2017a). Learning from these earthquake events, this earthquake study of Bengkulu Province is focused on Bengkulu City. Large earthquake events have not only happened in Bengkulu Province, but also in many other provinces in Indonesia, such as Nangroe Aceh Darussalam in 2004, North Sumatra in 2005, and West Sumatra in 2009. Those earthquakes also triggered massive structural damage, which revealed that the seismic design code in Indonesia needs to be evaluated. Considering these earthquake events, the Indonesian Government revised the previous seismic design code (SNI 03-1726-2002) to a new one (SNI 03-1726-2012). The updated

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seismic design code is now becoming the reference for local engineers in Indonesia for construction design (Mase & Somantri, 2016), and should be considered as design practice for buildings all over Indonesia.

In this study, a seismic response analysis due to earthquakes is presented. The study aims to check the reliability of the updated seismic design code (SNI 03-1726-2012) with regard to earthquakes in Bengkulu. The ground motion is propagated from the surface of engineering bedrock through the soil layer. At the ground surface, it is further transferred to the spectral acceleration curves, which are compared to the updated seismic design code. In addition, the amplification factor between the propagated ground motion and the ground motion analyzed at the ground surface is also presented. In general, the study is expected to provide better understanding of seismic ground response analysis in Bengkulu City, as well as to provide suggestions to local engineers in consideration of spectral acceleration design for construction design in Bengkulu City.

2. STUDY AREA AND GENERAL GEOLOGICAL CONDITIONS

The study area is located along the coastal area of Bengkulu City (Figure 1), where during the 2000 and 2007 earthquakes large-scale destruction took place. In the study, standard penetration test (SPT) and spectral analysis of surface wave (SASW) test were conducted, and information from the tests interpreted to establish the geological conditions of the investigated area. In general, the sub-soil in the area was dominated by sandy soils. Loose sand classified as SP was found at a depth range of 0 to 1.5 m, and 7.5-9 m, with $(N_1)_{60}$ of 5-6 blows/ft and FC (fines content) of 4-7%. Medium sand classified as SM existed at a depth range of 1.5 to 22.5 m, with $(N_1)_{60}$ of 15-25 blows/ft and FC of 10 to 18%. At a depth range of 22.5 to 30 m, dense sand layers classified as SC and SM were found. In terms of soil resistance, $(N_1)_{60}$ on these layers ranged between 25-35 blows/ft, with FC of 16-22%. According to the National Earthquake Hazard Reduction Program, or NEHRP (1998), in general site classification of the study area is categorized as stiff soil (site class type D), with V_{S30} (average of V_S up to 30 m deep) of 298 to 302 m/s.

3. METHOD OF ANALYSIS

3.1. Seismic Hazard Parameter

Peak ground acceleration (PGA) prediction is very important in earthquake risk analysis. This parameter is normally estimated by the attenuation model, which corresponds to the earthquake source mechanism. In Bengkulu, there are two major earthquake sources, which have triggered many intensive earthquake events: shallow crustal sources, which include active tectonic fault earthquakes and stable continental region earthquakes; and the subduction zone, which includes intraplate and interplate earthquake events.

In this study, several attenuation models (Table 1), which are designed to estimate the peak ground acceleration at the bedrock of these earthquake mechanisms, are employed to determine the PGA parameter. In Table 1, for the shallow crustal earthquake mechanism occurring in the active tectonic fault region, Next Generation Attenuation (NGA) models are used, including those of Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014) and Chiou and Youngs (2014). For earthquakes occurring in the stable continental region, the attenuation models used are those of Dahle et al. (1995), Hwang and Huo (1997), Toro (2002) and Pezeshk et al. (2011). The Atkinson and Boore (2003), Kataoka et al. (2006), Shoushtari et al. (2016) and Idini et al. (2017) models are used to predict the PGA for earthquakes under a subduction mechanism. These attenuation models for the earthquake source mechanism consider earthquake uncertainty, such as magnitude, source distance, and site effect, which vary due to the geotechnical and geological aspects.

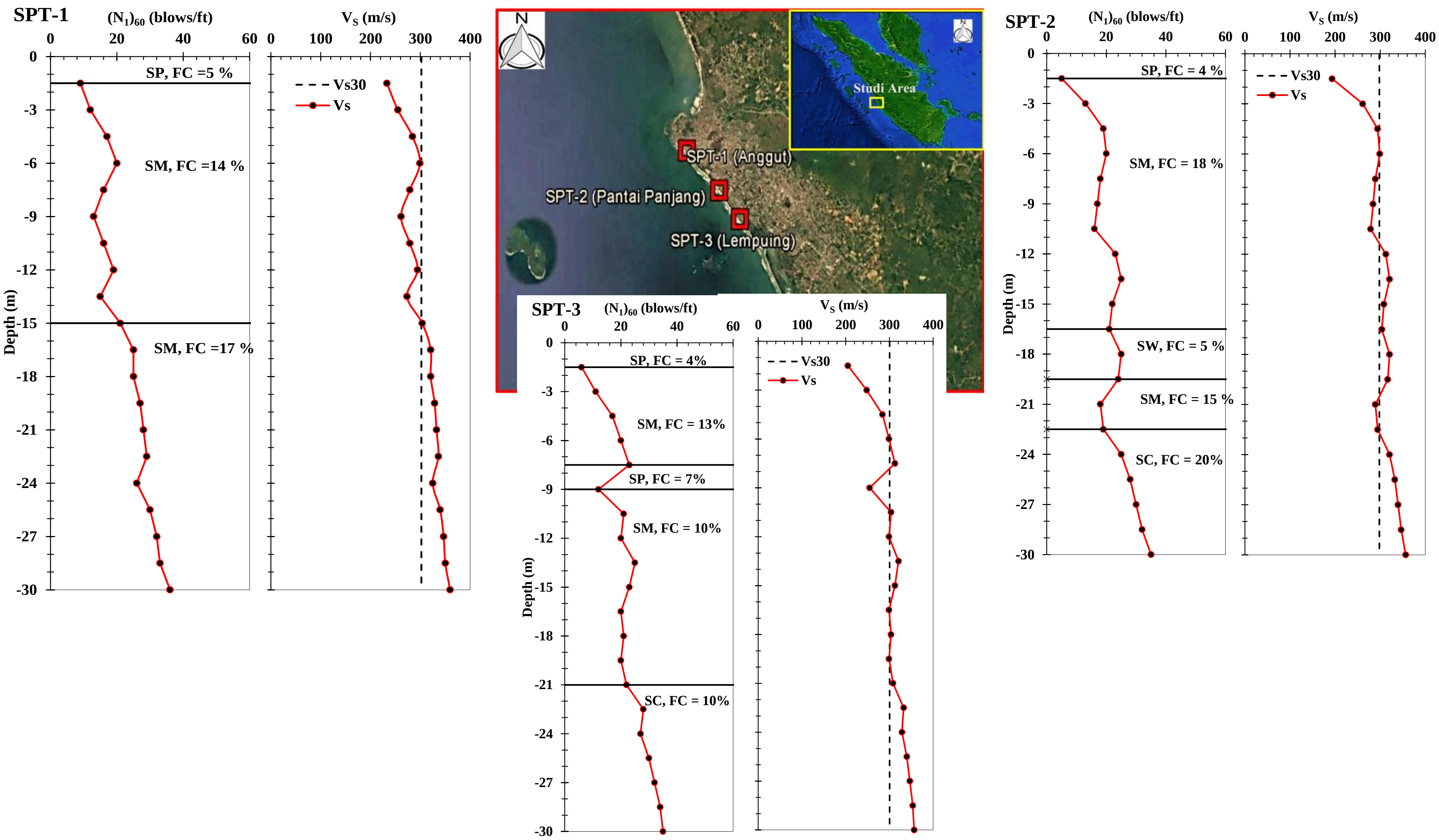


Figure 1 Study area and site investigation results

3.2. Deterministic Seismic Hazard Analysis

Seismic Hazard Analysis is normally performed to predict the risk level of earthquake impact, which includes probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) (Reiter, 1990). PSHA is a statistical method used to predict a representative earthquake, considering its recurrence in a region (Kinasih et al., 2014), whereas DSHA is a simple deterministic method which determines the most credible earthquake or the controlling earthquake. In DSHA, the most credible earthquake is predicted based on the serious impact on the study area (Mase & Somatri, 2016).

Table 1 Attenuation models used in the study

Active Tectonic Region	Stable Continental Region	Subduction Zone
Abrahamson et al. (2014)	Dahle et al. (1995)	Atkinson & Boore (2003)
Boore et al. (2014)	Hwang & Huo (1997)	Kataoka et al. (2006)
Campbell & Bozorgnia (2014)	Toro (2002)	Shoushtari et al. (2016)
Chiou & Youngs (2014)	Pezeshk et al. (2011)	Idini et al. (2017)

The impact on the site is defined by the Modified Mercalli Intensity (MMI) scale proposed by Wood and Neuman (1931). In this study, a total of 1,968 earthquake events in Bengkulu Province zone are analysed by the DSHA method. The data are collected from Mase's (2010) study and the National Agency for Meteorology, Climatology and Geophysics, or BMKG (2017), website. These earthquakes occurred in 2000-2016. During this period, seismic activity in Bengkulu Province increased significantly. Among the 1,968 events, the significant earthquakes occurring each year are selected. These earthquakes are called the representative earthquakes, and are plotted in Figure 2.

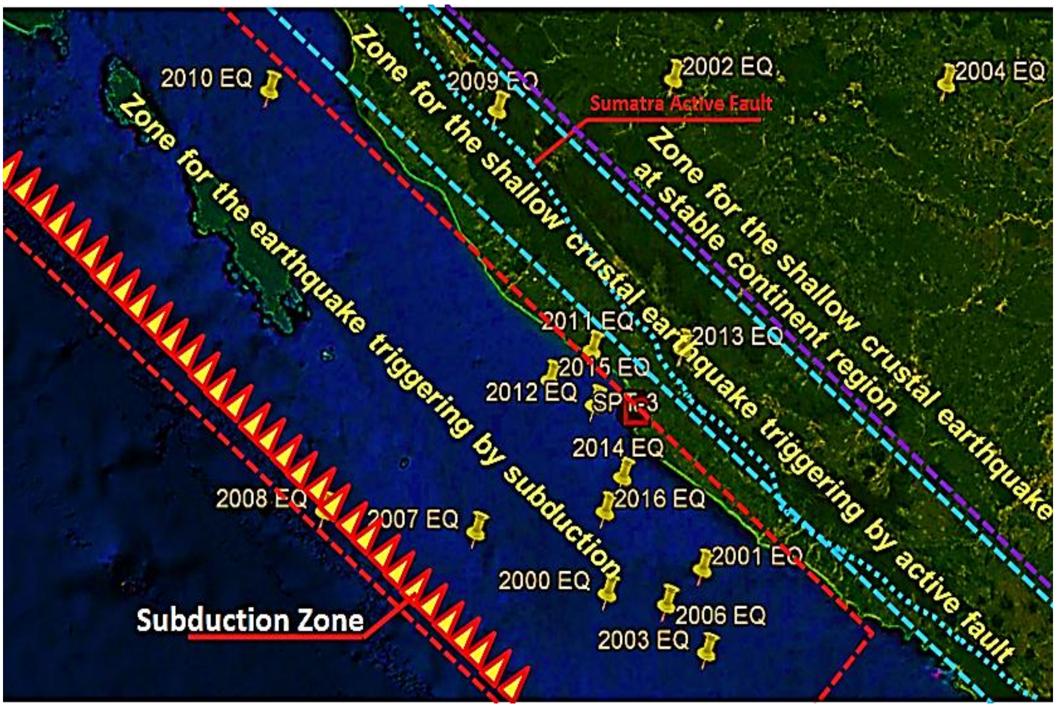


Figure 2 Representative earthquakes each year (modified from Google Earth, 2017)

In Figure 2, the earthquake zones corresponding to the source mechanism are depicted; i.e. the subduction zone, active fault zone, and stable continental zone. From this zonation, estimation of the earthquake source can be determined. Table 2 compiles the representative earthquakes from the period 2000 to 2016 and presents the source mechanism and type of earthquake.

Among the representative earthquakes, the 2009 and 2013 events are categorised as shallow crustal earthquakes, being active tectonic region type. Both earthquakes occurred due to activity in the Sumatra Fault (locally, the Semangko Fault). Two other shallow crustal earthquake events occurred in 2002 and 2004, and are also categorized as earthquakes at stable continental region, whereas the other remaining representative earthquakes are categorized as subduction earthquakes. For the earthquakes which occurred under subduction activity, the 2012 and 2016 events occurred in the intraface zone (focal depth > 50 km), whereas the others occurred in the interface zone (focal depth < 50 km). By using all the information provided in Figures 1 and 2 and Tables 1 and 2, attenuation model analysis is performed. Furthermore, among these earthquakes, the most destructive earthquake from the 2000-2016 period is determined. This earthquake is defined as the controlling earthquake, considered to be that which had the most significant impact.

Table 2 Maximum PGA of the representative earthquakes for the investigated zones

Year	M_w	Earthquake Mechanism	Type	PGA at the Investigated Points (g)			Attenuation Model
				SPT-1	SPT-2	SPT-3	
2000	7.9	Subduction	Interplate	0.1553	0.1586	0.1607	Atkinson-Boore (2003)
2001	7.2	Subduction	Interplate	0.1214	0.1232	0.1239	Shoustari et al. (2016)
2002	5.7	Shallow Crustal	Stable Continent	0.0123	0.0122	0.0122	Dahle et al. (1995)
2003	5.7	Subduction	Interplate	0.0296	0.0303	0.0308	Shoustari et al. (2016)
2004	7.1	Shallow Crustal	Stable Continent	0.0155	0.0155	0.0155	Dahle et al. (1995)
2005	6.2	Subduction	Interplate	0.0184	0.0183	0.0185	Shoustari et al. (2016)
2006	5.8	Subduction	Interplate	0.0407	0.0416	0.0428	Shoustari et al. (2016)
2007	8.6	Subduction	Interplate	0.2121	0.2119	0.2110	Idini et al. (2017)
2008	6.6	Subduction	Interplate	0.0517	0.0509	0.0501	Shoustari et al. (2016)
2009	6.7	Shallow Crustal	Active Tectonic	0.0075	0.0155	0.0151	Chiou & Youngs (2014)
2010	4.4	Subduction	Interplate	0.0020	0.0019	0.0019	Shoustari et al. (2016)
2011	5.3	Subduction	Interplate	0.0897	0.0885	0.0870	Shoustari et al. (2016)
2012	5.2	Subduction	Intraplate	0.0331	0.0331	0.0329	Shoustari et al. (2016)
2013	4.2	Shallow Crustal	Active Tectonic	0.0053	0.0053	0.0052	Abrahamson et al. (2014)
2014	3.9	Subduction	Interplate	0.0111	0.0116	0.0118	Shoustari et al. (2016)
2015	4.8	Subduction	Interplate	0.0293	0.0289	0.0283	Shoustari et al. (2016)
2016	5.8	Subduction	Intraplate	0.0626	0.0626	0.1042	Shoustari et al. (2016)
Maximum PGA (PGA_{max}) of all earthquakes over 17 years				0.2121	0.2119	0.2110	"Controlling earthquake"

Tables 2 and 3 show the PGA of the attenuation model analysis of the investigated sites (SPT-1 to SPT-3) and the scale of MMI caused by the representative earthquakes. In the study, all the attenuation models from the relevant sources are used to calculate PGA. Furthermore, from the calculation results, only the highest value is considered for the next step of the analysis, i.e. selecting the controlling earthquake, as presented in Table 2. In this table, the maximum value of PGA on each site and the attenuation model resulting from the maximum PGA between all the relevant models are presented. Based on Table 2, two big earthquakes resulted in the highest impact on the site. The earthquakes occurred in 2000, a 7.9 M_w earthquake with a PGA average of 0.158g, and in 2007, the 8.6 M_w earthquake with a PGA average of 0.217g; both were triggered by subduction zone activity. In Table 3, the MMI earthquake scale is presented. This scale is derived from Tjokrodimuljo's (2000) method, by which it can be estimated by log(PGA)=(MMI scale/3). The MMI scales for the earthquakes are VIII and IX respectively. Both earthquakes had the potential to result in very serious damage, as reflected by very high levels of MMI. A detailed description of the MMI scale can be found in Wood and Neumann (1931). In line with the results, the 2007 earthquake was selected as the controlling earthquake event in the study area.

Table 3 Maximum MMI of the representative earthquakes for the investigated zones

Year	Magnitude (M_w)	PGA at the Investigated Points (g)			Modified Mercalli Intensity (MMI)		
		SPT-1	SPT-2	SPT-3	SPT-1	SPT-2	SPT-3
2000	7.9	0.1553	0.1586	0.1607	VIII	VIII	VIII
2001	7.2	0.1214	0.1232	0.1239	VIII	VIII	VIII
2002	5.7	0.0123	0.0122	0.0122	V	V	V
2003	5.7	0.0296	0.0303	0.0308	VI	VI	VI
2004	7.1	0.0155	0.0155	0.0155	V	V	V
2005	6.2	0.0184	0.0183	0.0185	V	V	V
2006	5.8	0.0407	0.0416	0.0428	VI	VI	VI
2007	8.6	0.2121	0.2119	0.2110	IX	IX	IX
2008	6.6	0.0517	0.0509	0.0501	VII	VII	VII
2009	6.7	0.0075	0.0155	0.0151	IV	IV	IV
2010	4.4	0.0020	0.0019	0.0019	II	II	II
2011	5.3	0.0897	0.0885	0.0870	VII	VII	VII
2012	5.2	0.0331	0.0331	0.0329	VI	VI	VI
2013	4.2	0.0053	0.0053	0.0052	IV	IV	IV
2014	3.9	0.0111	0.0116	0.0118	V	V	V
2015	4.8	0.0293	0.0289	0.0283	VI	VI	VI
2016	5.8	0.0626	0.0626	0.1042	VII	VII	VII

3.3. Input Motions

To reduce the uncertainty of the observed locations, determination of ground motions based on those of the different sources was conducted. Since this study aims to make seismic response analysis and compare this to the Indonesian Seismic Design Code (SNI 03-1726-2012), the regulations in the design code need to be considered, especially the minimum number of analysed ground motions (at least five). In the study, the five source ground motions used were:

1) Loma Prieta Earthquake, 18 October 1989, 090 CDMG Station 47381
2) Imperial Valley Earthquake, 15 October 1979, USGS Station 5115
3) Kobe Earthquake, 16 January 1995, Kakogawa CUE90 Station
4) Northridge Earthquake, 17 January 1994, 090 CDMG Station 24278
5) Chichi Earthquake, 20 September 1999, TCU045 Station

Furthermore, the source ground motions were scaled corresponding to the maximum PGA of each investigated site (SPT-1, SPT-2 and SPT-3), obtained from the attenuation model analysis. Examples of the scaled source ground motions are shown in Figure 3.

3.4. One-dimensional Seismic Response Analysis

3.4.1. Soil behavior under dynamic load

One-dimensional non-linear seismic response analysis based on the non-linear finite element approach was conducted. In the model, soil non-linearity simulated by incremental plasticity was performed to observe deformation and damping, as well as excess pore water pressure, if liquefaction is also considered. This method was developed by Lu et al. (2006), and is based on the effective stress concept (Ishihara et al., 1975; Parra, 1996; Yang, 2000) combined with the multi-yield surface framework proposed by Prevost (1985). In this method, the incremental stiffness is evaluated, with emphasis placed on controlling the permanent strain deformation. An illustration of soil behavior in the model is presented in Figure 4.

3.4.2. Soil parameters

The main soil parameters used in the non-linear seismic response analysis included shear wave velocity (V_S), mass density (ρ), friction angle (ϕ), cohesion (c), Poisson's ratio (υ), effective mean confinement pressure (p'), coefficient of lateral earth pressure (K_o), peak shear strain (γ), and number of yield surfaces, amongst others. The parameters related to soil properties and field measurement were determined from laboratory tests and site investigation. For other specific parameters, assumptions were made. The list of parameters used in this study for each layer is summarized in Table 4.

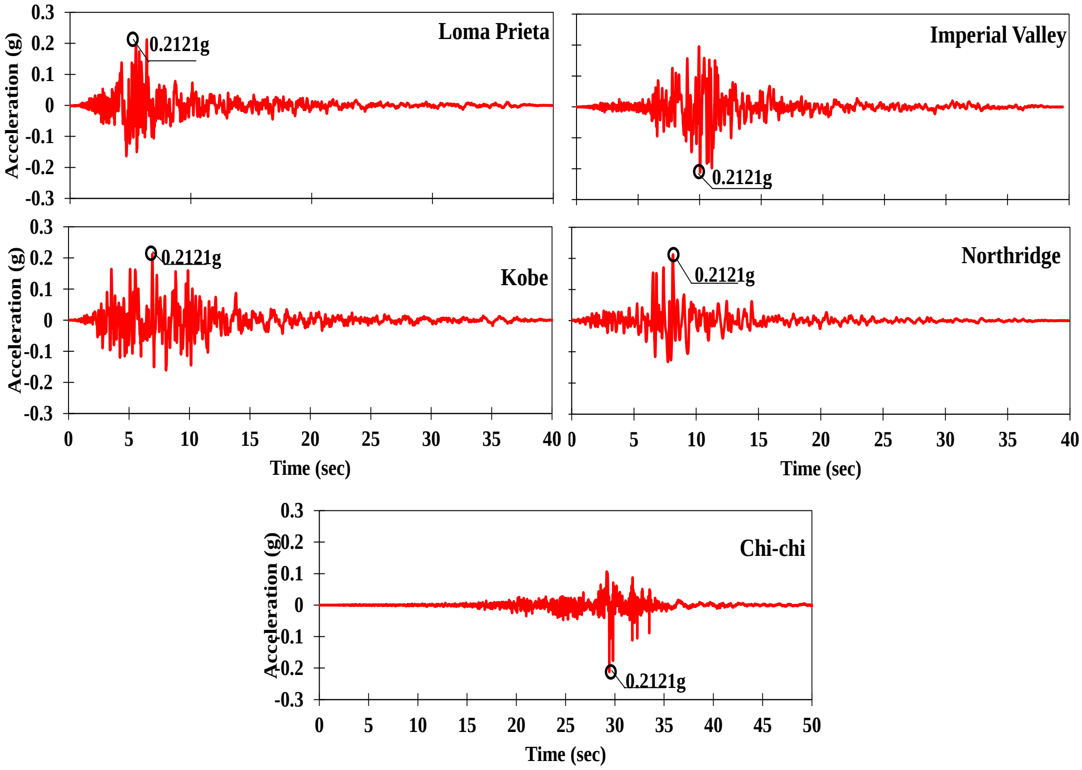
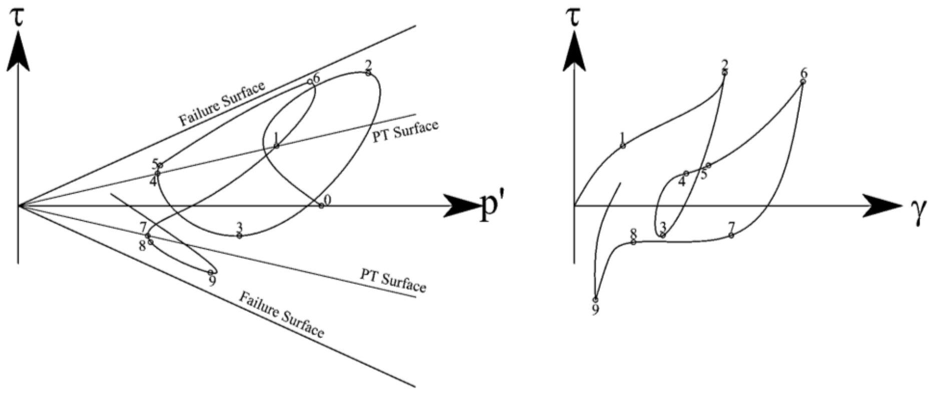
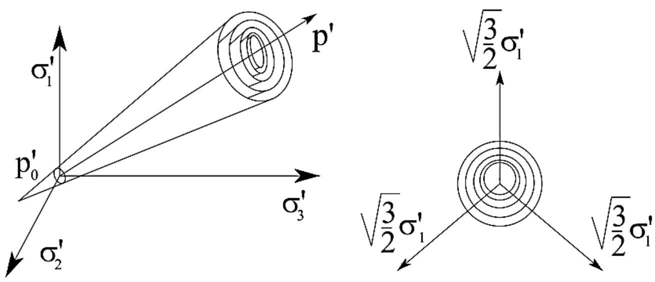


Figure 3 Examples of the scaled ground motions for SPT-1



(a)



(b)

Figure 4 Soil behavior under dynamic load: (a) effective stress path (adapted from Lu et al., 2006); (b) multi-yield surface mechanism (adapted from Lu et al., 2006)

The material properties (Table 4) of each layer were determined based on either undisturbed or disturbed sampling tests from the soil samples taken from boring test. Brief explanations of the material properties are given below (a detailed explanation of the soil parameters can be found in Lu et al., 2006).

1) ρ is soil density, FC is soil fines content, c is soil cohesion, ϕ is the internal friction angle of the soil, and h is layer thickness
2) V_S is average shear wave velocity for the soil layer
3) K_o is the coefficient of lateral earth pressure at rest, and P_{ref} is reference mean effective confinement
4) γ_{max} is peak shear strain, ν is Poisson's ratio, G is shear modulus, and K is bulk modulus.
5) The yield surface number is assumed to be 20, which is based on the recommendation provided in Lu et al. (2006).

In this study, since the depth of investigation is only 30 m below the ground surface, interpolation of shear wave velocity to the surface of the engineering bedrock is performed. As a result, the assumption of the material for the depth of 30 m to the engineering bedrock surface needs to be adjusted. In the analysis, at the bottom of the borehole, V_S is assumed to be 760 m/s, which means it is categorized as a soft rock material. A list of the material assumptions for the interpolated depth is also presented in Table 4.

Table 4 Input parameters used in the simulation

SPT	Layer	Soil Type	h (m)	ρ (kg/m3)	V_S (m/s)	P_{ref} (kPa)	K_o	c (kPa)	ϕ (°)	γ_{max} (%)	ν	FC (%)	G (kPa)	K (kPa)
1	Layer 1	SP	1.5	1760	116	100	0.515	2	29	5	0.4	5	23842	111263
	Layer 2	SM	13.5	1900	281	100	0.470	5	32	5	0.4	14	149967	699844
	Layer 3	SM	15	2100	336	100	0.470	6	32	5	0.4	17	237067	1106311
	Layer 4	Assumed	20	2200	555	100	0.470	6	32	5	0.4	17	677655	3162390
2	Layer 1	SP	1.5	1750	97	100	0.531	2	28	5	0.4	4	16389	76483
	Layer 2	SM	15	1910	265	100	0.500	5	30	5	0.4	18	134005	625357
	Layer 3	SW	3	2120	319	100	0.485	1	31	5	0.4	5	215437	1005374
	Layer 4	SM	3	1900	292	100	0.515	7	29	5	0.4	15	161844	755272
	Layer 5	SC	7.5	2100	339	100	0.470	10	32	5	0.4	20	241691	1127893
	Layer 6	Assumed	20	2200	553	100	0.470	10	32	5	0.4	20	672780	3139639
3	Layer 1	SP	1.5	1720	102	100	0.515	1	29	5	0.4	4	18062	84291
	Layer 2	SM	6	1905	286	100	0.500	3	30	5	0.4	13	155767	726913
	Layer 3	SP	1.5	1700	127	100	0.531	1	28	5	0.4	7	27589	128751
	Layer 4	SM	12	1900	306	100	0.485	4	31	5	0.4	10	177671	829131
	Layer 5	SC	9	2100	343	100	0.470	9	32	5	0.4	10	246930	1152339
	Layer 6	Assumed	20	2200	553	100	0.470	9	32	5	0.4	10	672780	3139639

3.4.3. Modelling criteria

The modelling criteria for the one-dimensional seismic response analysis are depicted in Figure 5. The input motion is applied at the bottom of the soil profile. To ensure that this motion propagates from the bedrock to the ground surface, information on the sediment thickness needs to be obtained. In this study, the estimation of the sediment thickness of the study area (the engineering bedrock surface) was considered based on the passive (microtremor) measurement provided in Refrizon et al.'s (2013) study, i.e. about 50 m deep. As the soil profile in this study is only 30 m, the soil profile is linearly interpolated by adjusting the V_S at 50 m equal to 760 m/s (NEHRP, 1998). The interpolating method applied to adjust the soil profile based on V_S is presented in Figure 6. In the model, the boundary condition is limited in the vertical direction. However, displacements on both vertical and horizontal directions are allowed. If liquefaction analysis is considered, excess pore water pressure can also be observed. This is because there is no drainage path in the lateral direction. The soil column is underlain by the impermeable elastic half space. To determine mesh size, wavelength analysis is made. For the study, a mesh size of 0.5 m was selected, as suggested by Pender et al. (2016), Mase et al. (2016), Mase et al. (2018a), and Mase et al. (2018b).

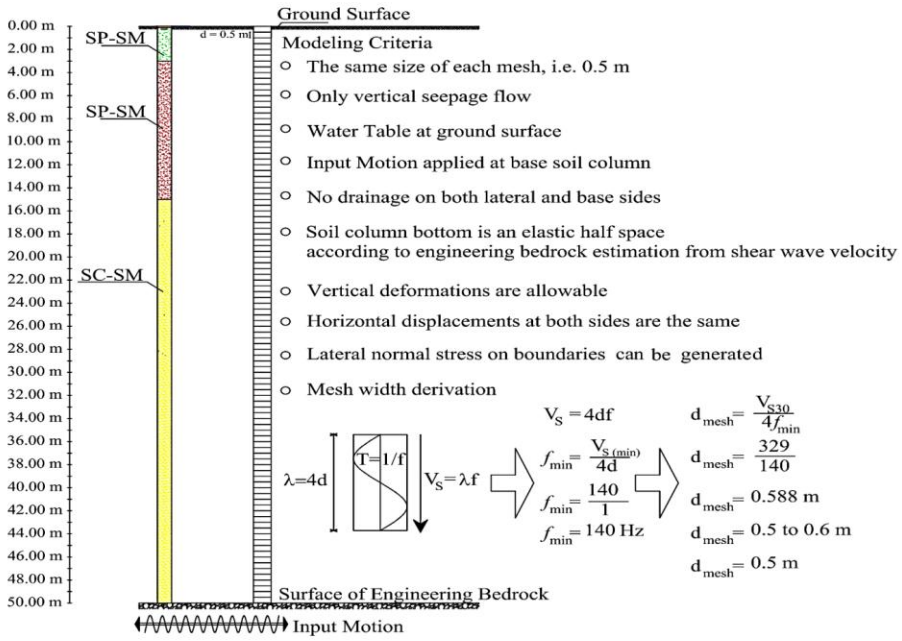


Figure 5 Illustration of one-dimensional seismic analysis (Adapted from Mase et al., 2017)

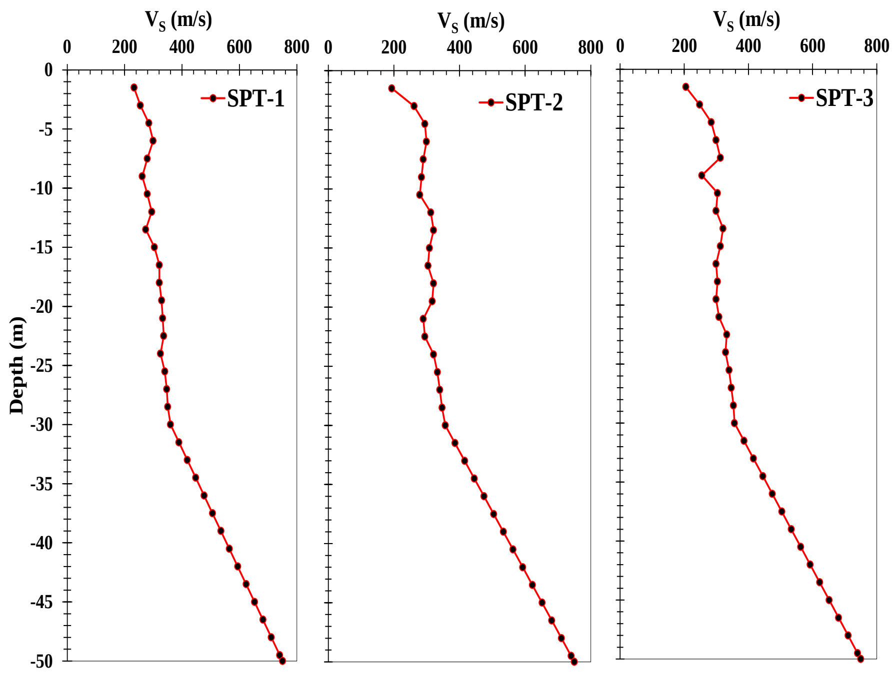


Figure 6 Soil profile interpolation for depths below the investigated layer

3.5. Spectral Acceleration of SNI 03-1726-2012

The spectral acceleration of the one-dimensional analysis was compared to the updated spectral acceleration design of SNI 03-1726-2012. This spectral acceleration was also developed based on seismic hazard analysis. The code includes three soil site criteria used in establishing the spectral acceleration for the earthquake load. These are classified as soft soil (SE), medium soil (SD) and stiff soil (SC) spectral accelerations. The soil site type can be predicted based on the value of soil resistance, such as $(N_1)_{60}$ for the first 30 m of depth and V_{S30}. The designed earthquake of SNI 03-1726-2012 was assigned as the earthquake with a 2% probability of exceedance in 50 years. The design code specified a return period of 2,475 years.

4. RESULTS AND DISCUSSION

4.1. Maximum Acceleration Profile and Amplification Factor

The interpretation of the maximum acceleration (PGA_{max}) profile is presented in Figure 7. In general, the input motion of each source ground motion experiences amplification on each site. The waves almost constantly propagate from a depth of 50 m to 35 m and start to amplify at 35 m up to ground level. Since linearization is applied at depths below 30 m, soil resistance increases with depth. This assumption does not seem to influence the wave propagation significantly. Moreover, at a depth of 30 m the sub-soil is dominated by dense sand, which may provide almost similar soil resistance to the assumed soil type at depths of 30 to 50 m. However, from a depth of 30 m to the ground surface, medium and loose sands dominate the sub-soils. The low resistance of sand, especially loose-medium sand, contributes to the amplification of the seismic propagation wave. Generally, the ground motion is amplified by about 1.9 to 2.7 times its initial value (Table 5); the Loma Prieta earthquake generated the highest amplification on each site, and the Chichi earthquake generated the lowest amplification.

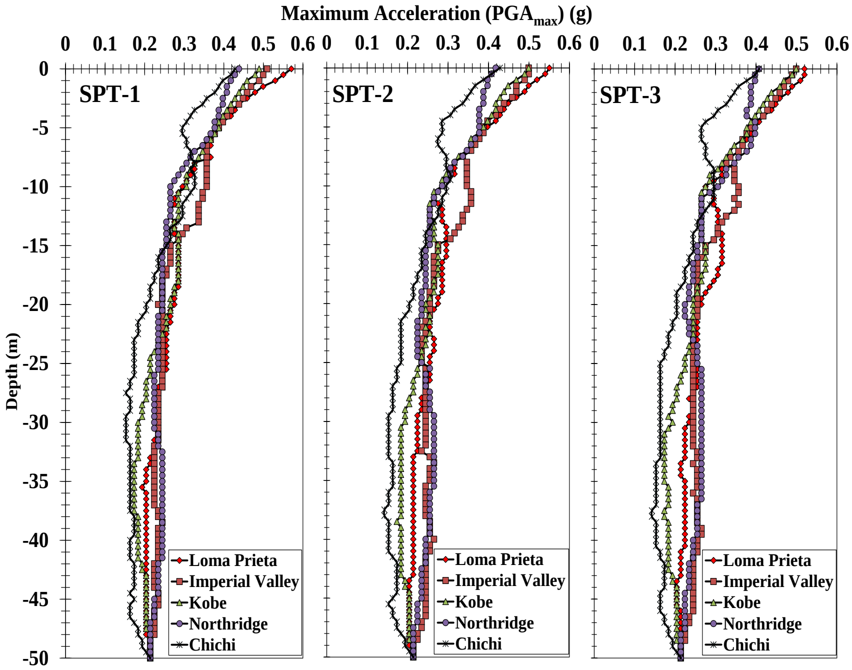


Figure 7 Maximum acceleration (PGA_{max}) profile resulting from the five source ground motions

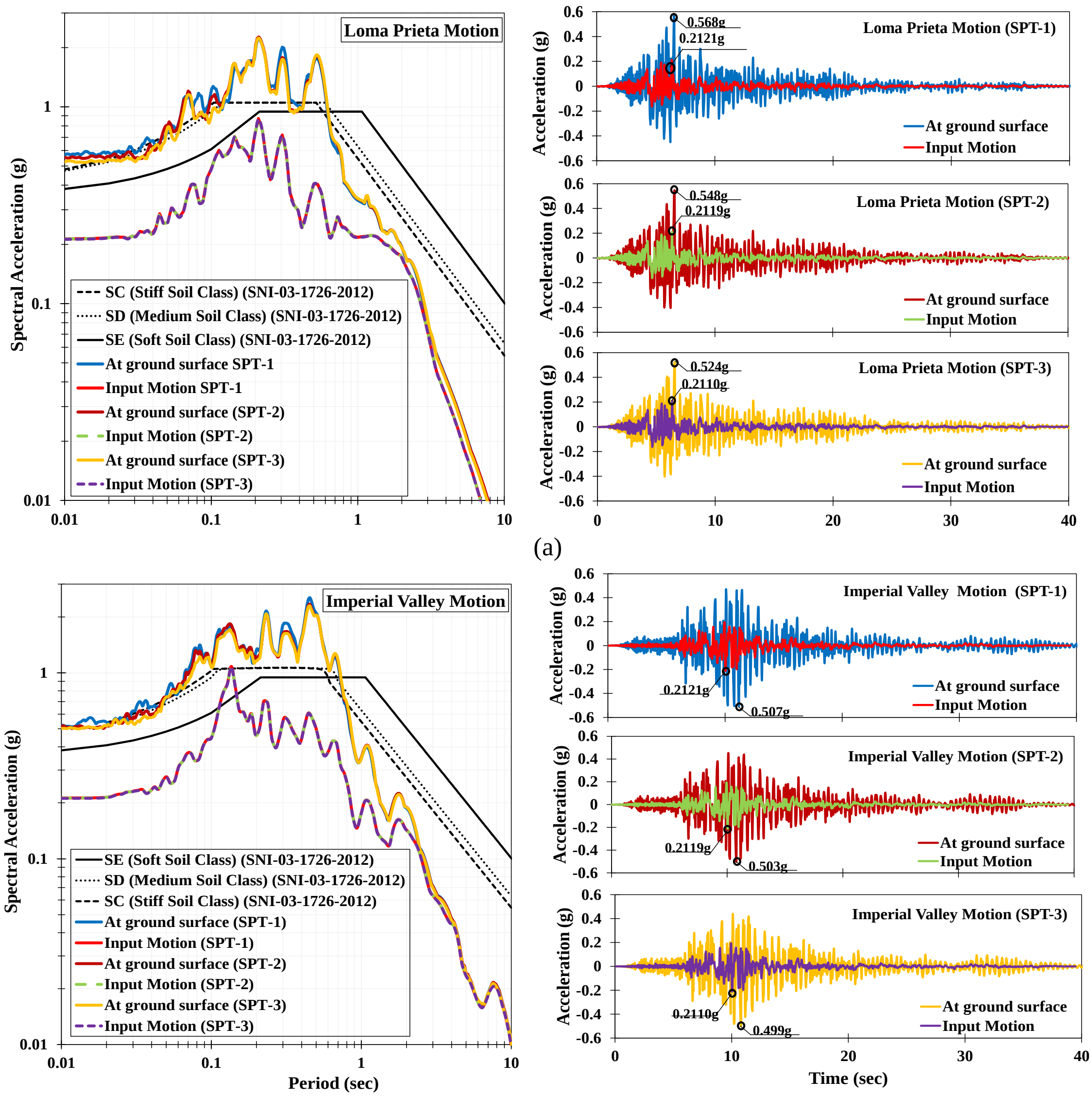
Table 5 Amplification factor of each investigated site

SPT-1				
Loma Prieta	Imperial Valley	Kobe	Northridge	Chichi
2.678	2.304	2.392	2.044	2.033
SPT-2				
Loma Prieta	Imperial Valley	Kobe	Northridge	Chichi
2.585	2.357	2.374	1.960	2.007
SPT-3				
Loma Prieta	Imperial Valley	Kobe	Northridge	Chichi
2.482	2.356	2.366	1.930	1.944

4.2. Spectral Acceleration Comparison

Comparison of the spectral acceleration resulting from the seismic response and the designed spectral acceleration at the ground surface is presented in Figure 8. In general, the spectral acceleration of seismic response analysis exceeds the spectral acceleration design for all site classes, especially on short period (Period (T_n) ≤ 0.2 s). In Bengkulu City, two story buildings are commonly found. The natural period of a building can be simply estimated by T_n = 0.1n, where n is the number of stories. Therefore, for a two-story building, T_n is 0.2 s.

In general, the results also show that for 2 to 7 story buildings (T_n of 0.2 to 1s), the spectral acceleration design is exceeded by the spectral acceleration of the seismic analysis for all soil sites. For long periods ($T_n \geq 1$), the designed spectral acceleration of all the sites is still sufficient to cover that of the seismic analysis. However, for the natural period of low to medium-rise buildings, this is a warning that Bengkulu, as developing city, may in the future construct many buildings with at least 5 to 7 stories, especially in those investigated locations where the social economy of the city is centralized. Based on this study, the earthquake aspect should be considered in building design in Bengkulu City.



(a)

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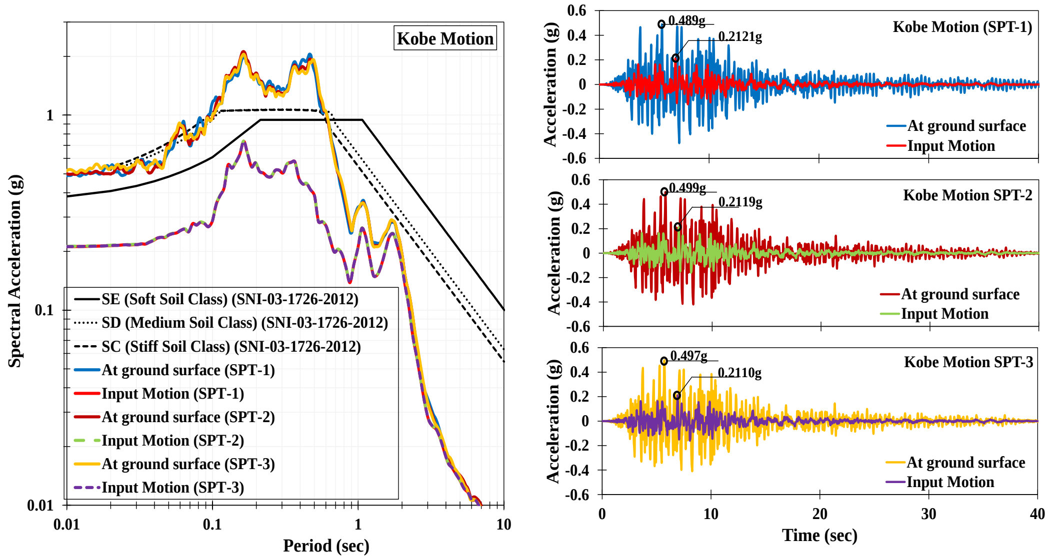

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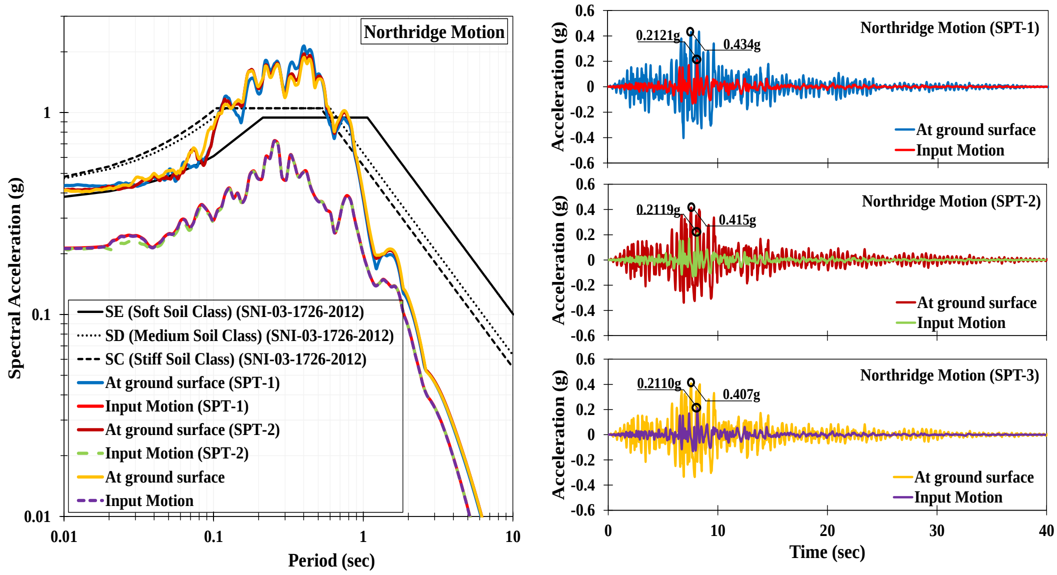
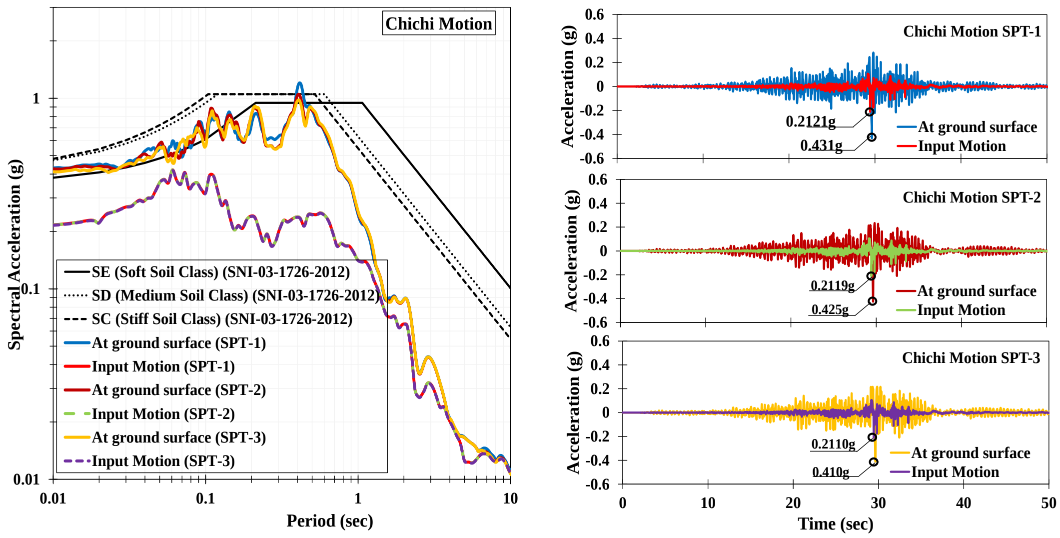


(d)



(e)

Figure 8 Spectral acceleration comparison: (a) Loma Prieta motion (b) Imperial Valley motion (c) Kobe motion (d) Northridge motion (e) Chichi motion

From the results, it can be concluded that the designed spectral acceleration released by SNI 03-1726-2012 is still able to cover that of the controlling earthquake, especially for long periods. However, buildings with $T_n \geq 1$s are still rare in Bengkulu City. This leads to the recommendation to conduct seismic response analysis before performing the structural analysis for building design in Bengkulu City, especially when designing 2 to 7 story buildings. In addition, the local government should be more careful in giving permission to construct buildings in this area, especially those that have not considered the specific earthquake load in the design aspect.

5. CONCLUSION

The results show that the designed spectral acceleration is still able to cover the spectral acceleration resulting from the wave propagation of the controlling earthquake. However, for $T_n \leq 0.2$s, 0.2s$\leq T_n \leq$ 0.7s, the designed spectral acceleration for three site classes (soft soil, medium soil and stiff soil) are exceeded by the spectral acceleration of the seismic response analysis. In addition, the great case needs to be taken when designing earthquake loads for the buildings. This study also brings a recommendation to study liquefaction in the investigated sites, since the sandy soils are generally found in the investigated sites. The numerical analysis as performed by Mase et al. (2017a) and the experimental study as performed by Mase (2017b) can be performed in the future.

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