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SITE RESPONSE ANALYSIS OF KLANG VALLEY SOIL UNDER DIFFERENT CONSTITUTIVE MATERIAL MODELS

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ABSTRACT

This paper presents an investigation on the influence of constitutive models on the seismic response of soil material under impact of earthquake loading. In this study, Site Response Analysis (SRA) is performed to evaluate the seismic response of two selected soil profiles located in Klang Valley area when subjected to three strong earthquake ground motion. The soil profiles are designed to behave under three constitutive models i.e., Mohr Coulomb (MC), Hardening Soil (HS), Hardening Soil Small (HSS). The numerical analyses are conducted using the PLAXIS 2D finite element software. The comparison on seismic deformation of the soil profiles highlights the important role of the constitutive model in modifying the seismic behaviour of the soil.

1.0 INTRODUCTION

Soil is a complicated material that responds to stresses by acting in a non-linear manner that regularly exhibits anisotropic and time-dependent behavior. Typically, soil behavior during primary loading, unloading, and reloading varies. It displays non-linear behavior well below failure conditions with stiffness that is affected by stress. Soil undergoes plastic deformation and is variable in dilatancy. Additionally, soil experiences minor strain stiffness at extremely low strains and upon stress reversal. This common behavior could not possibly be accounted for in simple elastic-perfectly plastic Mohr Coulomb model, although the model has advantages that make it an ideal option for a soil model. Constitutive models have developed over a period, from being simple to more complex to capture the behavior of soil under complex loading conditions. These models have been formulated based on the principles of continuum mechanics and numerical evaluation of the models with respect to the facility which can be implemented in computer calculations [1]. Constitutive models for simulation of soil behavior other than Mohr Coulomb are Hardening Soil, HS Small, Modified Cam-Clay etc.

Site response analysis (SRA) is an essential instrument in geotechnical and earthquake engineering employed to assess the response of soil strata to seismic waves as they propagate from bedrock to the surface. This analysis elucidates the impact of local soil characteristics on the amplitude, frequency, and duration of ground vibrations, which substantially affects the seismic performance of structures. Through site response analysis, engineers can ascertain site-specific ground motion attributes, facilitating enhanced seismic design and evaluation of structures such as buildings, bridges, tunnels, and other infrastructure [2 -6]. This paper discussed the influence of constitutive models on modifying the soil behavior and response under impact of earthquake loads. The study deals with soil-structure interaction under seismic loading conditions requires the use of a constitutive model to reproduce the seismic behavior of soils to evaluate

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the response of soil from small deformations to failure. To achieve the intended objectives, SRA was performed to evaluate the seismic response of two selected soil profiles located in Klang Valley area when subjected to three strong earthquake ground motion.

1.1 Mohr-Coulomb (MC)

Mohr-Coulomb model is an elastic-perfectly plastic model which is often used to model soil behaviour in general and serves as a first-order model. In general stress state, the model's stress-strain behaves linearly in the elastic range, with two main key parameters of the Hooke's law i.e., Young's modulus, E and Poisson's ratio, v [7]. Meanwhile, the other important parameters of this constitutive models are i) friction angle; ii) cohesiveness; and iii) dilatancy angle. These parameters allow the model to determine failure criterion and describes the use of a non-associated flow rule that is utilised to mimic a realistic irreversible change in volume due to shearing [7-8]. The flow rule serves as the evolution law for plastic strain rates in traditional plastic theory. If the plastic potential function is the same as the yield function, the flow rule is called the associated flow rule and it is different, it is called the non-associated flow rule. For simulating the behaviour in the area where negative dilatancy is prominent in soil mechanics, such as the Cam clay model for typically consolidated clay, associated flow rules have been employed. Whereas the non-associated flow rule typically describes the behaviour of sands with both positive and negative dilatancy.

Although drained circumstances represent failure behaviour adequately, the effective stress route taken by undrained materials may differ dramatically from observations. In an undrained analysis, it is desirable to utilise undrained shear parameters with a zero-friction angle. It is difficult to accurately describe the stiffness (and thus deformation) behaviour prior to the local shear. The strain hardening or softening impact of the soil is not considered in the concept for perfect plasticity. The Mohr-Coulomb yield condition is an extension of Coulomb's friction law to general states of stress. In fact, this condition ensures that Coulomb's friction law is obeyed in any plane within a material element. The full Mohr-Coulomb yield condition consists of six yield functions when formulated in terms of principal stresses as shown in Equations (1a) to (1f) [8-9]:

$$f_{1a} = \frac{1}{2}(\sigma_2' + \sigma_3') + \frac{1}{2}(\sigma_2' + \sigma_3')\sin(\varphi) - c\cos(\varphi) \leq 0 \tag{1a}$$

$$f_{1b} = \frac{1}{2}(\sigma_3' + \sigma_2') + \frac{1}{2}(\sigma_3' + \sigma_2')\sin(\varphi) - c\cos(\varphi) \leq 0 \tag{1b}$$

$$f_{2a} = \frac{1}{2}(\sigma_3' + \sigma_1') + \frac{1}{2}(\sigma_3' + \sigma_1')\sin(\varphi) - c\cos(\varphi) \leq 0 \tag{1c}$$

$$f_{2b} = \frac{1}{2}(\sigma_1' + \sigma_3') + \frac{1}{2}(\sigma_1' + \sigma_3')\sin(\varphi) - c\cos(\varphi) \leq 0 \tag{1d}$$

$$f_{3a} = \frac{1}{2}(\sigma_1' + \sigma_2') + \frac{1}{2}(\sigma_1' + \sigma_2')\sin(\varphi) - c\cos(\varphi) \leq 0 \tag{1e}$$

$$f_{3b} = \frac{1}{2}(\sigma_2' + \sigma_1') + \frac{1}{2}(\sigma_2' + \sigma_1')\sin(\varphi) - c\cos(\varphi) \leq 0 \tag{1f}$$

The two plastic model parameters appearing in the yield functions are the well-known friction angle φ and the cohesion c. The condition fi = 0 for all yield functions together where fi is used to denote each yield function which represents a fixed hexagonal cone in principal stress space [9] as shown in Figure 1.

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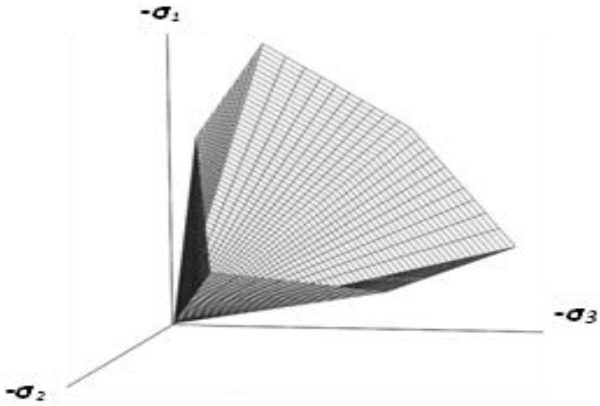

Figure 1. Mohr-Coulomb yield surface in principal stress space (c = 0) [9]

In addition to the yield functions, six plastic potential functions as listed in Equations (2a) to (2f) are defined for the Mohr-Coulomb models. The plastic potential functions contain a third plasticity parameter, the dilatancy angle ψ. This parameter is required to model positive plastic volumetric strain increments (dilatancy) as actually observed for dense soils.

$$g_{1a} = \frac{1}{2}\ (\sigma_2' + \sigma_3'\ ) + \frac{1}{2}\ (\sigma_2' + \sigma_3'\ )\sin(\psi) \leq 0 \tag{2a}$$

$$g_{1b} = \frac{1}{2}\ (\sigma_3' + \sigma_2'\ ) + \frac{1}{2}\ (\sigma_3' + \sigma_2'\ )\sin(\psi) \leq 0 \tag{2b}$$

$$g_{2a} = \frac{1}{2}\ (\sigma_3' + \sigma_1'\ ) + \frac{1}{2}\ (\sigma_3' + \sigma_1'\ )\sin(\psi) \leq 0 \tag{2c}$$

$$g_{2b} = \frac{1}{2}\ (\sigma_1' + \sigma_3'\ ) + \frac{1}{2}\ (\sigma_1' + \sigma_3'\ )\sin(\psi) \leq 0 \tag{2d}$$

$$g_{3a} = \frac{1}{2}\ (\sigma_1' + \sigma_2'\ ) + \frac{1}{2}\ (\sigma_1' + \sigma_2'\ )\sin(\psi) \leq 0 \tag{2e}$$

$$g_{3b} = \frac{1}{2}\ (\sigma_2' + \sigma_1'\ ) + \frac{1}{2}\ (\sigma_2' + \sigma_1'\ )\sin(\psi) \leq 0 \tag{2f}$$

1.2 Hardening Soil (HS)

The Hardening Soil model is a true second-order model for soils in general (soft soils as well as harder types of soil), for any type of application [10-11]. The model involves friction hardening to model the plastic shear strain in deviatoric loading, and cap hardening to model the plastic volumetric strain in primary compression. A distinction can be made between two main types of hardening, namely shear hardening and compression hardening.

Shear hardening is used to model irreversible strains due to primary deviatoric loading. Compression hardening is used to model irreversible plastic strains due to primary compression in oedometer loading and isotropic loading. Both types of hardening are contained in the present model. Failure is defined by means of the Mohr-Coulomb failure criterion. The model is accurate for issues involving a decrease in mean effective stress and concurrent mobilisation of shear strength due to the two types of hardening. Such circumstances can arise during excavation projects (retaining wall issues) and during tunnel construction. Some basic characteristics of the model are stressing dependent stiffness according to a power law (m), plastic straining due to primary deviatoric loading (Eref 50), plastic straining due to primary compression (Eref oed), elastic unloading/reloading input parameters (Eref ur, Vur) and failure criterion according to the Mohr-Coulomb model (c, φ and ψ) [8-9].

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The model accurately depicts a decrease in mean effective stress under undrained loading, as was the case for soft soils, but it also has the potential to depict an increase in mean effective stress for harder soil types (dilative soils). In a variety of geotechnical applications, this model can be used to precisely anticipate displacement and failure for common types of soil.

1.3 Hardening Soil with Small-Strain Stiffness (HSS)

The original Hardening Soil model assumes elastic material behaviour during unloading and reloading. However, the strain range in which soils can be considered truly elastic, i.e., where they recover from applied straining almost completely, is very small. With increasing strain amplitude, soil stiffness decays nonlinearly. Plotting soil stiffness against log(strain) yields characteristic S-shaped stiffness reduction curves. The characteristic shear strains that can be measured near geotechnical structures and the applicable strain ranges of laboratory tests [8-9]. It turns out that at the minimum strain which can be reliably measured in classical laboratory tests, i.e., triaxial tests and oedometer tests without special instrumentation, soil stiffness is often decreased to less than half its initial value.

1.4 Site Response Analysis (SRA)

Site effects are referred to as those intensities and frequency content changes in a specific seismic excitation due to the wave propagation characteristics of the subsoil and the topographic characteristics that have a direct impact on the structural response during an earthquake [11]. Bidimensional models are typically employed to study topographic influences, which change the soil seismic response; time-domain methods are most frequently employed to determine soil SRA. Near-source time-histories usually report important vertical accelerations; thus, it is recommended to evaluate the soil site response using SRA in 2D or 3D that may incorporate all three time-history components of an earthquake signal.

Recent studies are focused on enhancing site response analysis through the refinement of soil constitutive models, the integration of three-dimensional effects, and the examination of soil-structure interaction. Researchers are investigating high-performance computing to undertake advanced non-linear analyses on complex soil profiles, thereby yielding more dependable forecasts of seismic ground motions [12-15]. The site response analysis is a crucial tool in earthquake engineering, offering vital insights into the influence of local soil conditions on seismic waves. The selection of a constitutive model, analytical approach, and precise soil profile are critical for obtaining dependable results, hence enhancing seismic design and safety.

2.0 METHODOLOGY

The influence of constitutive model material on the seismic response of soil profile subjected to strong ground motion was investigated by performing the SRA for two soil profile. This procedure has been developed by modelling the two selected layered soil profiles located at Klang Valley using the PLAXIS 2D finite element software. The soil profiles are designed to behave under three constitutive models i.e., Mohr Coulomb (MC), Hardening Soil (HS), Hardening Soil Small (HSS). The details soil material properties are presented in Table 1 below. Three seismic input motions i.e., 1989 Loma Prieta Earthquake, 1995 Kobe Earthquake, and 1995 Hyougoken South Earthquake (Table 2) are adopted to highlight the influence of constitutive models on the seismic response of the soils. This decision was made in accordance with the recommendation of FEMA 356 [16]. In order to investigate the response of soil-tunnel models using the nonlinear time-history analysis methodology, a minimum of three time-history data is required [16].

2.1 Soil Profiles and Properties

Two soil profiles were taken into consideration in accordance with the constitutive models MC, HS, and HSS. The soil properties tabulated in Table 1 taken from the available soil profiles from the previous research and available soil investigation report. In particular, the soil model A represents the soil profile for underground Stormwater Management and Road Tunnel (SMART)[15], meanwhile the soil model B is the soil properties taken at underground location of Mass Rapit Transit (MRT) tunnel [17]. Both locations are located at Klang Valley, Malaysia. Each type of soil has distinct soil property criteria, ranging from stiff

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to soft ground media. Soil B is represented by SPT N value which indicates the hardness of soil profile. Whereas soil A is classified based on particle size distribution classification standard.

Table 1. Soil properties

Soil type	Model Soil A [15]			Model Soil B [17]		
	Clay	Silt	Sand	N<30	30<N<100	N>100
Material behaviour	Drained	Drained	Drained	Drained	Drained	Drained
Density (mass), γ (kN/m³)	18	18	20	18.5	19	20
Modulus of elasticity, *E* (N/mm²)	9	8	90	0.87(2N)	0.87(2N)	0.87(2N)
Friction angle, φ (˚)	25	25	31	28	28	29
At-rest earth pressure coefficient, K_0	0.4	0.5	0.92	0.8	0.8	0.8

2.2 Eearthquake Load

Three-time history data are selected as the minimum number of ground motions for dynamic analysis as recommended by FEMA 356 [16] for the earthquake hazard's motion assessments. Real-time histories taken from PEER [10] was chosen as the input motion for the free field analysis with peak ground accelerations (PGA) ranging from 0.288g to 0.781g (see Table 2 and Figure 2).

Table 2. Selected real earthquake records [18]

Earthquake	Moment magnitude, Mw	Epicentral distance, (km)	PGA (g)	Duration (s)
Loma Prieta, California, 1989	6.9	96	0.288	39.99
Kobe, Japan, 1995	6.9	20	0.452	32.00
Hyougoken South, Japan, 1995	7.2	17	0.781	30.00

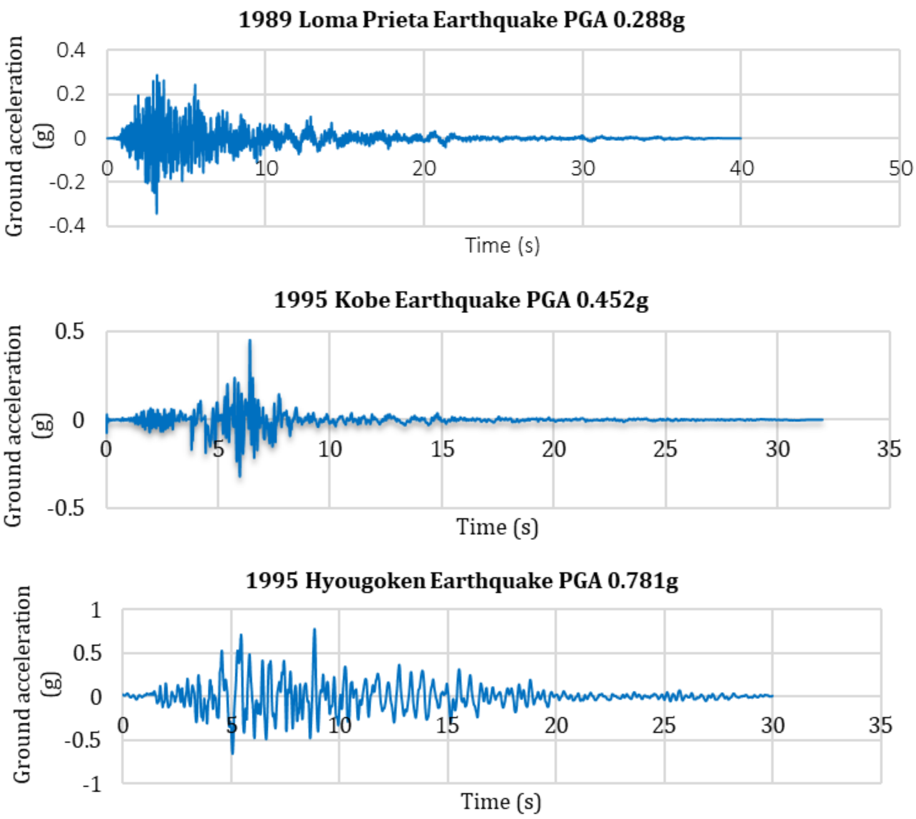

Figure 2. Three real of time-history records of strong ground motions Adapted from PEER Ground Motion Database [18]

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2.3 Numerical Modelling

This study models the geometrical shape and dimensions of soil and tunnel according to the recommendations of the British Tunnelling Society (2004) in the publication 'Tunnel Lining Design Guide,' as well as databases from the Mass Rapid Transit Corporation Sdn. Bhd. (MRTC) and MMC Gamuda KVMRT (PDP-SSP) Sdn. Bhd. for the MRT Putrajaya Line. The geometrical dimensions of the 2D soil-tunnel model were established as 100 meters in the x-direction and 45 meters in the y-direction, with a tunnel diameter of 7 meters. The tunnel lining was located at a burial depth of 30 meters, measured from the ground surface to the crown of the tunnel lining.

The adopted mesh dimension size ensures the efficient reproduction of all waveform frequencies in the simulated tunnel models. Layers of soil profiles are shown in Figure 3. It was assumed that Vs increments linearly in the function of the effective overburden (σ’v0) [18] and that the local materials have a linear behaviour and a damping ratio of 2%, to consider the non-linear subsoil behaviour it was employed shear stiffness (G/Gmax) and damping ratio curves (λ) as in Figure 4.

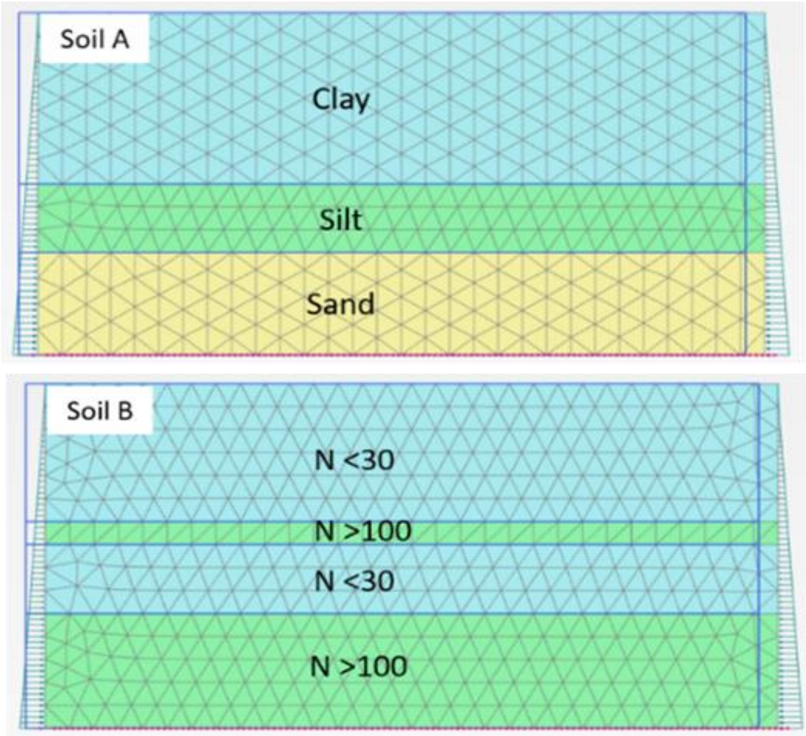

Figure 3. Layers of two (2) soil profile

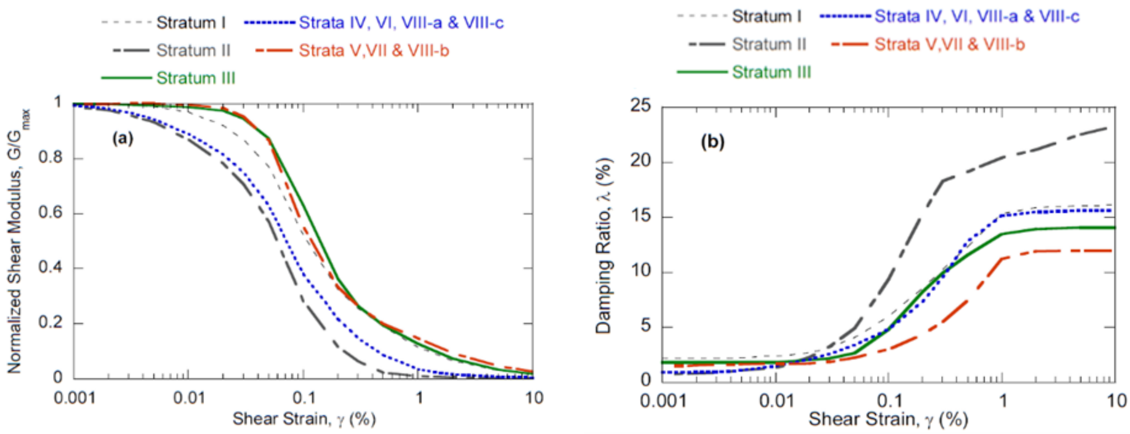

Figure 4. (a) Normalized shear stiffness G/G_{max} and (b) Damping ratio

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3.0 RESULT AND DISCUSSION

In this section, total of 18 cases (Table 5) of Site Response Analysis (SRA) were conducted to investigate the influence of constitutive models i.e., Mohr Coulomb (MC), Hardening Soil (HS) and Hardening Soil Small (HSS) on the seismic response of soil material under impact of three selected real strong earthquake loading. The site response analysis results indicate that maximum soil deformation increases with the peak ground acceleration (PGA) of the applied earthquake loading. In Soil Profile A, the deformation during the Hyogoken earthquake, which had a PGA of 0.781g, was considerably greater (0.4212×10^{-3} m) for the Mohr Coulomb model) than the deformation observed during the Loma Prieta earthquake with a PGA of 0.288g (0.5474×10^{-6} m) for the same model. A comparable pattern was noted in Soil Profile B, indicating that increased PGA events led to enhanced soil deformation. The findings are consistent with prior research, Kramer (1996) [2] that has demonstrated a direct correlation between seismic intensity and soil deformation.

A comparative analysis of the two soil profiles indicates that Soil Profile B consistently exhibits greater maximum soil deformation than Soil Profile A under the same loading conditions. In the case of the Kobe earthquake, utilising the Hardening Soil model, the maximum deformation observed in Soil Profile A was $0.06649\times10-3$ m, whereas in Soil Profile B, it rose to 0.1739×10^{-3} m. Soil Profile B demonstrates increased deformability, probably attributable to its reduced stiffness or enhanced compressibility, which are critical geotechnical factors affecting seismic response [19]. The choice of the constitutive material model significantly influences the predicted soil deformation. The Mohr Coulomb (MC) model consistently yielded the lowest deformation predictions, demonstrating its limited responsiveness to nonlinear soil behaviour. The Hardening Soil Small (HSS) model yielded the highest deformation estimates due to its consideration of small-strain stiffness degradation and its advanced representation of nonlinear soil behaviour. For the Kobe earthquake in Soil Profile A, the HSS model predicted a deformation of 0.02796×10^{-3} m, while the MC model predicted 0.04142×10^{-3} m. This finding highlights the significance of employing advanced models such as HSS for precise predictions, especially in contexts with high seismic loads [11].

The implications of these results are significant for seismic design. Soil Profile B demonstrates increased deformation, and areas exposed to elevated PGA values necessitate improved design strategies to reduce the risks of significant soil movement and possible structural damage. The application of advanced constitutive models, specifically Hardening Soil (HS) and Hardening Soil Small (HSS), is advisable for accurate predictions of soil behaviour during seismic loading conditions. The numerical findings of this investigation are summarised in Table 5. Meanwhile, the numerical simulation of the soil, modelled using the MC and HS constitutive material models under the influence of the Hyogoken earthquake (PGA 0.781g), is presented in Figures 5 and 6, respectively.

Table 5. Result of Site Response Analysis

Soil profile	Earthquake Loading	Constitutive Material Model	Maximum soil Deformation (m)
Soil profile A	Loma Prieta PGA 0.288g	Mohr Coulomb (MC)	0.5474×10^{-6}
		Hardening Soil (HS)	0.4769×10^{-5}
		Hardening Soil Small (HSS)	0.3235×10^{-3}
	Kobe PGA 0.452g	Mohr Coulomb (MC)	0.04142×10^{-3}
		Hardening Soil (HS)	0.06649×10^{-3}
		Hardening Soil Small (HSS)	0.02796×10^{-3}
	Hyougoken PGA 0.781g	Mohr Coulomb (MC)	0.4212×10^{-3}
		Hardening Soil (HS)	0.2976×10^{-3}
		Hardening Soil Small (HSS)	0.02796×10^{-3}
Soil profile B	Loma Prieta PGA 0.288g	Mohr Coulomb (MC)	0.06731×10^{-3}
		Hardening Soil (HS)	0.02852×10^{-3}
		Hardening Soil Small (HSS)	0.02304×10^{-3}
	Kobe PGA 0.452g	Mohr Coulomb (MC)	0.1703×10^{-3}
		Hardening Soil (HS)	0.1739×10^{-3}
		Hardening Soil Small (HSS)	0.03453×10^{-3}
	Hyougoken PGA 0.781g	Mohr Coulomb (MC)	0.1337×10^{-3}
		Hardening Soil (HS)	0.03846×10^{-3}
		Hardening Soil Small (HSS)	0.02330×10^{-3}

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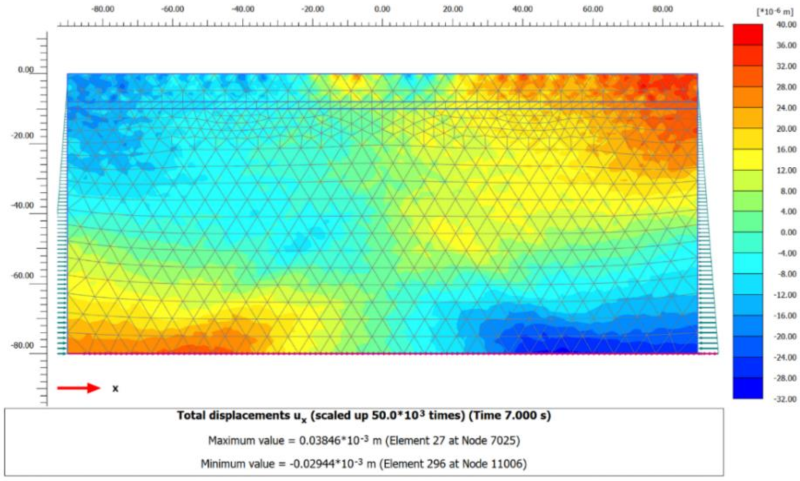

Figure 5. Total displacement of soil simulated using the MC material model during the 1995 Hyogoken Earthquake

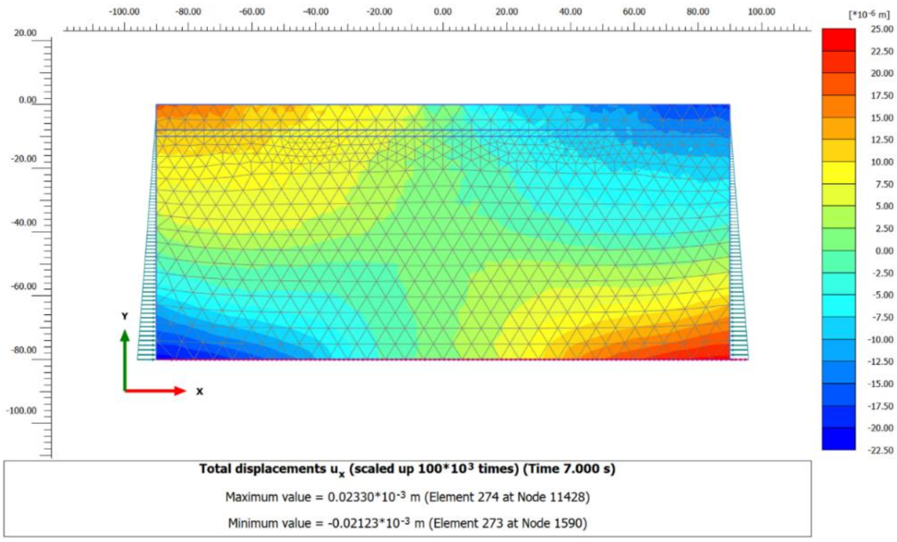

Figure 6. Total displacement of soil simulated using the HS material model during the 1995 Hyogoken Earthquake impact.

4.0 CONCLUSION

The site response study demonstrates the significant impact of earthquake loading intensity, soil profile attributes, and material constitutive models on maximum soil deformation. The findings demonstrate that an increase in peak ground acceleration (PGA) correlates with a rise in soil deformation, with greater deformation noted during more severe seismic occurrences, such as the Hyogoken earthquake. Soil Profile B repeatedly shown greater deformation than Soil Profile A, indicating its reduced stiffness or increased compressibility, which are critical elements in earthquake reactivity. The Mohr Coulomb model underestimated deformation due to its restricted consideration of nonlinear soil behaviour, while the Hardening Soil Small model offered more accurate predictions by integrating small-strain stiffness degradation and advanced soil properties.

These findings highlight the necessity of employing suitable constitutive models and comprehensive site-specific soil characterisation in seismic design. The findings underscore the necessity for supplementary measures to enhance the resilience of structures in regions characterised by high seismicity and softer soil conditions, as these are more susceptible to significant deformation.

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6.0 RECOMMENDATION

Based on the findings of this study, several recommendations are proposed to improve the accuracy of seismic response analysis and promote the resilience of structures in earthquake-prone regions. The recommendations pertain to the vital components of modelling precision, site-specific data acquisition, design factors, and possible research opportunities:

i. Application of Advanced Constitutive Models: It is advisable to employ advanced constitutive models, specifically the Hardening Soil and Hardening Soil Small models, for forthcoming seismic analyses. These models yield more accurate predictions of soil behaviour under dynamic loading by include stiffness degradation and nonlinear soil behaviour.

ii. Comprehensive Geotechnical Investigations: Site-specific geotechnical characterization should be prioritized to ensure accurate representation of soil properties. This is especially crucial for projects situated in regions with soft or highly compressible soils, which demonstrate increased vulnerability to seismic-induced deformations.

iii. Seismic Design Enhancements: Infrastructure in areas with elevated seismic activity or on deformable soil profiles must include improved seismic design strategies. Ground improvement methods, foundation isolation, or energy dissipation systems must be evaluated to alleviate excessive soil deformation and guarantee structural integrity.

iv. Validation using Experimental Data: Numerical analyses must be corroborated by experimental data, such large-scale shaking table tests or centrifuge modelling. This validation would strengthen confidence in the precision and dependability of the constitutive models employed in seismic analysis.

v. Creation of Seismic Fragility Curves: To enhance the assessment of seismic risk, fragility curves for various soil profiles must be established. These curves can incorporate numerical and experimental findings to evaluate susceptibility and inform the execution of suitable mitigation actions.

These recommendations aim to address the limitations observed in the analysis and provide a pathway for improving seismic performance assessment in future studies and practical applications.

7.0 CONFLICT OF INTEREST

The authors declare no conflicts of interest.

8.0 AUTHORS CONTRIBUTION

Wijaya Mohammad Yusof, B. M. W. (Conceptualization; Literature review; Writing - original draft)
Che Osmi, S. K. (Writing - critical revision of the article for important intellectual content
Mohd Fazully, M. F. I. (Simulation work; Writing- contributed to manuscript drafting)
Hafizi, N. (Simulation work; Writing- contributed to manuscript drafting)

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List of References

[1] Phillips, C., & Hashash, Y. M. A. (2009). Damping formulation for nonlinear 1D site response analyses. *Journal of Soil Dynamics and Earthquake Engineering, 29*, 1143-1158.
[2] Kramer, S. L. (1996). Geotechnical Earthquake Engineering. Prentice Hall.
[3] Rathje, E. M., Faraj, F., Russell, S., & Bray, J. D. (2005). Empirical relationships for frequency content parameters of earthquake ground motions. Earthquake Spectra, 21(2), 471-485.

*Corresponding Author | Che Osmi, S. K. | sitikhadijah@upnm.edu.my

[4] Pitilakis, K., Riga, E., & Anastasiadis, A. (2012). Design spectra and amplification factors for Eurocode 8. Bulletin of Earthquake Engineering, 10(5), 1377-1400.
[5] Konthesingha, C. P., Wijesooriya, M. S., & Ratnayake, R. M. C. (2020). Comparative assessment of constitutive soil models in seismic response analysis. International Journal of Geotechnical Engineering, 14(2), 151-160.
[6] Rahmani, H., & Pasha, S. (2021). Evaluating soil behavior under seismic loading using advanced constitutive models. Soil Dynamics and Earthquake Engineering, 140, 106375.
[7] Smith, I. M., & Griffiths, D.V. (1982). Programming the Finite Element Method. John Wiley & Sons, Chisester, U.K, 2nd Ed.
[8] Chen, C. F. (1985) "Mechanics of Geomaterials", In: Z. Bazant; editor. John Wiley & Sons Ltd. pg 65-86.
[9] PLAXIS Material Models Manual, CONNECT Edition V21.0.
[10] Brinkgreve, R. B. J. (2005) "Selection of soil models and parameters for geotechnical engineering application". In: Yamamuro, JA and Kaliakin, VN, editors. Geotechnical Special Publication No.128, ACSE, pg 69-98.
[11] Brinkgreve, R. B. J., Swolfs, W. M., & Engin, E. (2007). *PLAXIS: Finite Element Code for Soil and Rock Analyses*. Balkema Publishers.
[12] Stewart, J. P., Afshari, K., & Hashash, Y. M. A. (2014). Guidelines for performing seismic site response analysis using downhole array data. Pacific Earthquake Engineering Research Center (PEER) Report 2014/16, University of California, Berkeley.
[13] S. K. Che Osmi and S. M. Ahmad. "Seismic fragility curves for shallow circular tunnels under different soil conditions". World Academy of Science, Engineering and Technology, London. International Journal of Civil, Environmental, Structural, Construction and Architectural Engineering, vol. 10, no. 10. (2016): 1281-1287.
[14] S. K. Che Osmi, S. M. Ahmad and A. Adnan. "Seismic fragility analysis of underground tunnel buried in rock". International Conference on Earthquake Engineering and Seismology, Germany. (2015).
[15] Adnan, A. Ramli, M. Z., Vahed, Y. K., & Che Osmi, S. K. (2016). Seismic performance of tunnel lining under long distant earthquake effects. *World Academy of Science, Engineering and Technology. International Science Index, Civil and Environmental Engineering, 10*(10).
[16] Federal Emergency Management Agency. (2000). *Prestandard and commentary for the seismic rehabilitation of buildings* (FEMA 356). Washington, DC: Federal Emergency Management Agency.
[17] MMC Gamuda Kvmrt Sdn. Bhd. (2017). Geotechnical Interpretative Report for Tunnelling. Projek Mass Rapid Transit Lembah Kelang Laluan Sungai Buloh – Serdang – Putrajaya. Unpublished technical report.
[18] Pacific Earthquake Engineering Research (PEER) Center, "PEER Ground Motion Database," 2022. Retrieved from https://ngawest2.berkeley.edu/
[19] Idriss, I. M., & Boulanger, R. W. (2008). *Soil Liquefaction during Earthquakes*. Earthquake Engineering Research Institute (EERI).

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