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The Influence of Effective Confining Pressure on Site Response Analyses

Mahdy Khari, Khairul Anuar Bin Kassim and Azlan Bin Adnan
Faculty of Civil Engineering, Universiti Teknologi Malaysia, 81310 Skudai, Johor Barhu, Malaysia

Corresponding Author: Mahdy Khari, TB0302-Crystal Tower-Jalan Changkat Indah 2-Bukit Indah-Ampang Selangor-68000, Malaysia Tel: +60-1724813678

ABSTRACT

Ground motions are highly coupled with site conditions. Dynamic soil properties such as shear modulus and damping are the most prominent factors of amplification phenomenon during earthquakes. The shear modulus degradation and damping ratio curves have been developed based on the nonlinearity soil from hysteresis loops. Effective confining pressure and plasticity index are governing factors that effect the nonlinearity soil behavior. In this study, the shear modulus degradation and damping ratio curves were derived from cases. The first case was based on only effective confining pressure, whereas, the second case was based on effective confining pressure and plasticity index. These two models adopted equivalent linear method for the site response analyses. A set of outcomes are introduced to show the influence of the two models of curves in the site response.

Key words: Equivalent linear method, shear modulus, shear wave velocity, effective confining pressure, plasticity index

INTRODUCTION

As it is illustrated in Fig. 1, the changes of the frequency content and the amplification of the earthquakes are caused by geologic conditions (Phillips and Hashash, 2009; Khari and Bazyar,

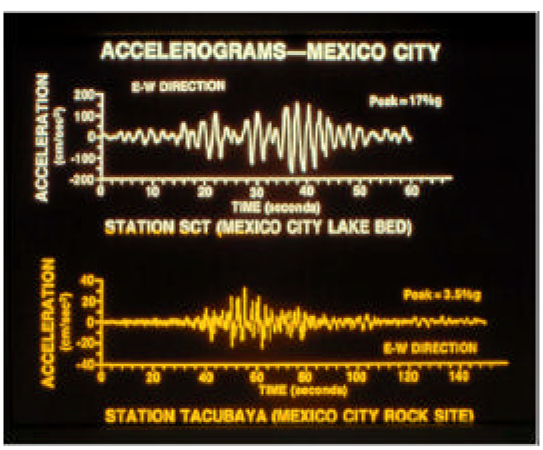

Fig. 1: The site effect on the 1985 Mexico City earthquake

2008) and the propagation of the seismic waves from the source of the earthquake to ground surface (Chavez-Garcia and Tejeda-Jacome, 2010).The researchers classify approaches to estimate the site effects in different types such as numerical and experimental. In the experimental method, the amplification is estimated through the analyses of the seismic registrations (Zhang *et al.*, 2005). In this approach, data of the amplification occurred during earthquakes of the past thirty years, are collected, then based on these data the local transfer function is determined (Nakamura, 1989). In the numerical methods, the soil properties are required to evaluate the site response analysis. Methods of analyzing the site response have been divided into one-dimension (1D), two-dimensions (2D) and three-dimension (3D). It is mentionable, some of the numerical methods are obviously in a testing phase, if the numerical and the experimental results are compared together, our astuteness about the site response analysis will be more trustworthy. Due to simplicity, now-a-days, one dimensional (1D) site response analysis methods are widely used in the design offices. In this paper, a model of the soil deposit is presented to determine the influence of the soil conditions to the wave propagation in the Penang Second Crossing (PSC). The shear velocity is computed by the downhole tests. The one-dimensional computer code is used in the frequency-domain, where hysteresis loops are created by the Masing (1926). The results of this soil model can be used in the soil-pile interaction. The stress-strain relationships of Ishibashi and Zhang (1993) are used to model the soil sediment.

THE EQUIVALENT LINEAR METHOD

One-dimensional methods can be divided into two groups: frequency-domain and time-domain analysis. Frequency domain analysis is consisting of different methods. The Equivalent Linear Method (ELM) is the most widely used among the researchers even if soil exhibits a non-linearity behavior. The ELM method, the soil is simulated based on Kelvin-Voigt model so that the shear strain can be calculated as follows:

$$\tau = G\gamma + \eta\dot{\gamma} \tag{1}$$

where, G is the shear modulus, η is the velocity, u (z, t) is horizontal displacement and $\dot{\gamma}$ is the shear strain rate:

$$\dot{\gamma} = \frac{\partial^2(u,t)}{\partial z.\partial t}, \quad \gamma = \frac{\partial(u,t)}{\partial z} \tag{2}$$

In case of harmonic loadings, the shear stress becomes:

$$\tau\,(z,t) = G^*\,(z,t) \tag{3}$$

where, G^* is the complex shear modulus ($G^* = G\,(1+2i\xi)$) and ξ is defined as the critical damping ratio (material), which can be determined based on Fig. 2. The Kelvin-Voigt model is modified in the ELM due to some of soil nonlinearities such as the shear modulus and the material damping ratio. As shown in Fig. 2, in the ELM, the secant shear modulus ($G = G_{sec}$) is assumed as the shear modulus. The nonlinearity stress-strain relationship indicates that there is arise in the damping ratio with increasing of strain amplitude. Variations G_{sec} and ξ with γ are defined by degradation curves. The degradation curves, in the different types of the soils, are presented by many researchers such as Seed and Idriss (1970), Seed *et al.* (1986), Sun *et al.* (1988) and Vucetic and Dobry (1991). It is worthy to note that the extra assumptions are used in this method about the frequency influences on stress-strain relations (Schanbel *et al.*, 1972). In the ELM, the wave

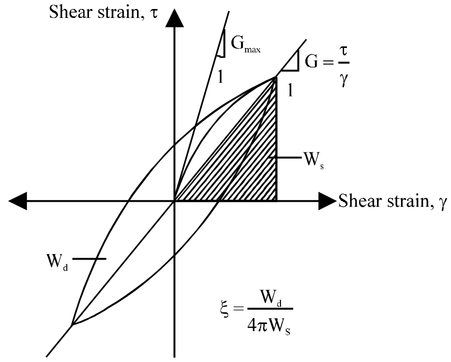

Fig. 2: Illustration of shear modulus and damping

equation is solved by the principle of superposition that it is valid for linear systems. For this reason, the nonlinearity inelasticity soil behavior can be simulated in equivalent linear approach with an effective level of shear strain (Υ_{eff}) induced in the soil. The effective shear strain is determined by the following equations:

$$\gamma_{eff} = R_r\, \gamma_{max} \tag{4}$$

$$R_r = \frac{M-1}{10} \tag{5}$$

where, R_r is strain reduction ratio, M is earthquake magnitude. The shear modulus and the damping ratio, for site response analysis, can be computed by measuring the shear velocity in site and variations of G and ξ with γ in laboratory. Both of the mentioned two ways are expensive, recently, a series of studies have been carried out for verifying variations of G and ζ with γ.

SOIL MODEL

As mentioned earlier, G and ζ are the main two factors in the site response analysis. These factors depend mostly on the effective mean confining stress ($\bar{\sigma}_o$) and plasticity index (I_p) so that several closed-form expressions have been developed by numerous researchers (Vucetic and Dobry, 1991; Sanchez-Sesma, 1987). However, some of these relationships are based on only one of the main two factors: (I_p) or ($\bar{\sigma}_o$), while the influences both of the two factors have been recognized by researchers (Ishibashi and Zhang, 1993). In 1993, the unified formulas G and ζ were developed by Ishibashi and Zhang (1993). As Fig. 3a shows at the higher levels of the effective mean confining pressure that the wave propagation has less energy dissipation. In Fig. 3b, it can be seen that the confining pressure effect is less than the plasticity index. The G/G_{max} degradation curve can be expressed:

$$\frac{G}{G_{max}} = k(\gamma, I_P)\,\bar{\sigma}_0^{\,-m(\gamma, I_P)-m_0} \tag{6}$$

where, k and m are the reduction and increase functions of the cyclic shear strain amplitude, respectively.

$$k(\gamma, I_P) = 0.5\{1+\tanh[\mathrm{In}[(\frac{\{0.000102+n(I_p)\}^{0.492}}{\gamma})]]\}^{1} \tag{7}$$

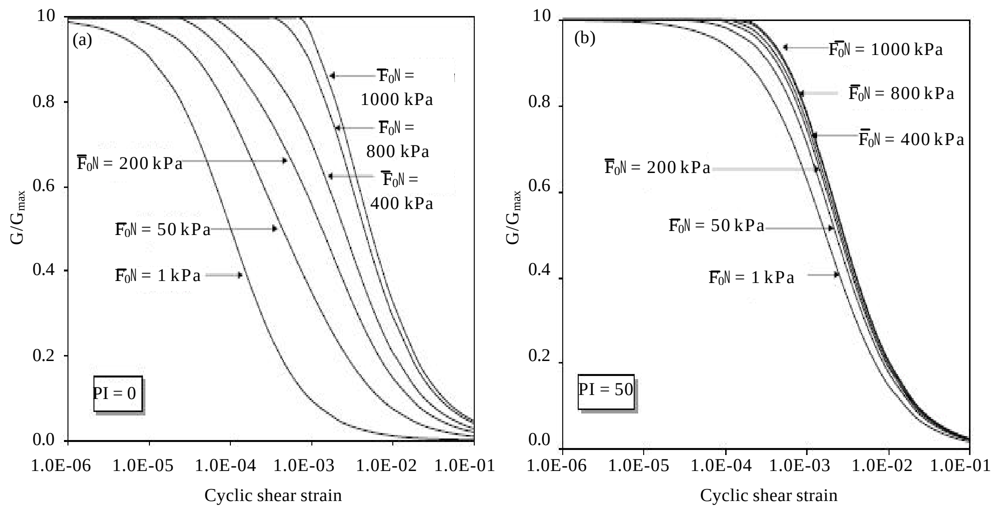

Fig. 3(a-b): Influence of effective confining pressure on modulus reduction curves: (a) Non-plasticity soil and (b) Plasticity soil (Ishibashi and Zhang, 1993)

$$m(\gamma,\ I_P)-m=0.272[1-\tanh[\ln\{\left(\frac{0.000556}{\gamma}\right)^{0.4}\}]e^{-0.0145\,IP^{1.3}} \tag{8}$$

The influence of the plasticity index is defined by n (I_P) that can be computed by the following equation:

$$n(I_P)=\begin{cases}0 & \text{for} \quad I_p=0\\ 3.37\times10^{-6}\,I_p^{1.404} & \text{for} \quad 0<I_p\le15\\ 7.00\times10^{-7}\,I_p^{1.976} & \text{for} \quad 15<I_p\le70\\ 2.70\times10^{-5}\,I_p^{1.115} & \text{for} \quad 70<I_p\end{cases} \tag{9}$$

The backbone curve can be plotted based on Eq. 3 and the loading-unloading behavior of soil can be simulated by the Masing (1926). The damping ratio can be calculated for the plastic and non-plastic soils as follows:

$$\xi=(\frac{0.333(1+e^{-0.01451P^{1.3}})}{2})\{0.586(\frac{G}{G_{max}})^2-1.547(\frac{G}{G_{max}})+1\} \tag{10}$$

REFERENCE SUBSOIL MODELS

The studied deposit was located in the Penang Second Crossing (PSC) at the North-West rejoin of Malaysia. The PSC is built from Batu Kawan in Mainland to Batu Maung in Penang Island. The PSC spans are sited on the two piers that are supported with the pile foundations. The piles length is equal to the 20 m. However, the piers length is different along the bridge due to the Penang Island topography. A series of laboratory and field tests were done by Soil Center Lab Sendirian Berhad in 2006. By noting the results of the tests, it was recognized that the soil region is consist of clay and sandy-clay mostly. The borehole 9 (BH-9) was selected for analyzing the site response, because the tests were done completely in the BH-9. The soil profile of the BH-9 is shown in Table 1. The downhole test was done for measuring the shear wave velocity. To evaluate the effective confining pressure in the site response analysis, the two models of the modulus degradation curves were entered in a computer program developed based on the equivalent linear site response. In the first case, Seed and Sun's modulus degradation and damping curves were used (case 1). While, in the second case, the modulus degradation and damping curves were calculated using equations 4 and 8 (case 2). The Sumatra Island's earthquakes were selected as the input motion and scaled to 0.08 g. As Fig. 4 shows the peak acceleration in the earthquake 1(EQ-1) and the earthquake 2 (EQ-2) are 0.22 and 0.13 g, respectively.

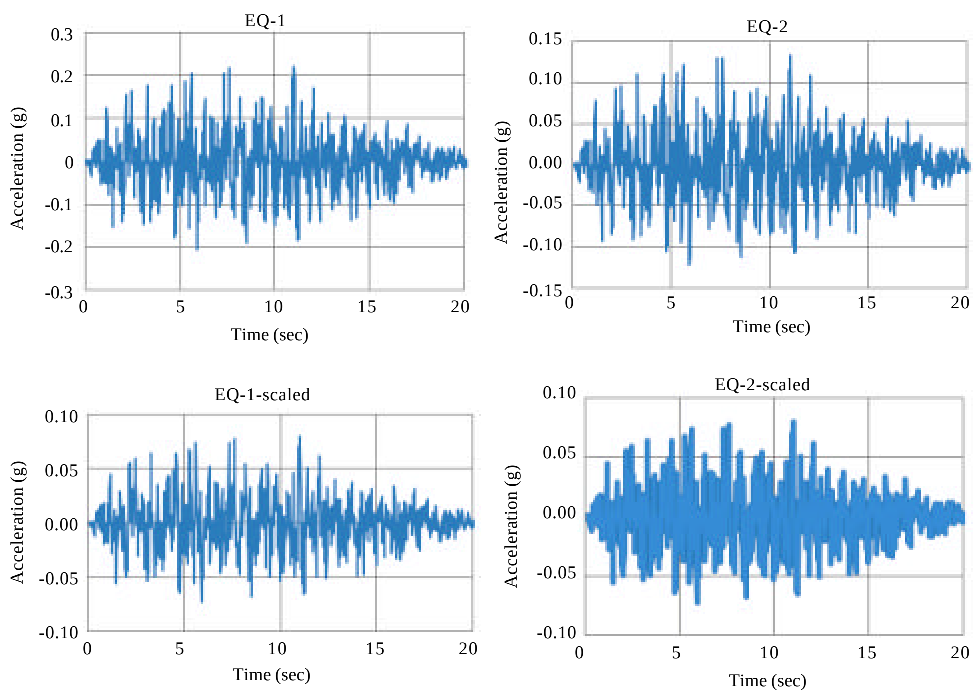

Fig. 4: Acceleration time histories used in the analyses

Table 1: Soil profile in BH-9

Soil	H (m)	ρ (kN m^{-3})
Clay	33	14.6
Sand	1.5	15.0
Sand	7.5	15.0
Clay	3	15.1
Sand	3	15.2
Clay	3	15.4
Sand	4.5	15.5
Clay	8	17.0
Sand	1.5	17.2
Clay	10	18.5

RESULTS

The analysis results were compared using the acceleration spectral, the amplification ratio, the acceleration time history at the surface and the maximum shear stress in the depth. Figure 5 shows that in the two earthquakes, while the maximum shear stress in case 1 is more than case 2 in depth but the values of the shear stress are equal to zero at the surface. In fact, the effective confining pressure was decreased to the surface and consequently, its influences cannot be negligible. Figure 6 illustrates the Max. acceleration occurred by the two ground motions in the depth. It can be shown, in the first case, the peak values of acceleration are remained almost stable from 80 to 15 m, Meanwhile, they are increased to 0.09 g at the surface. There is a gradual rise in the values of the Max. acceleration between the depth 80 and 55 m, in the case 2. From the depth 50 to 14 m, the amount of max accelerations is fluctuated. As it is represented in Fig. 7, the values of the Peak Ground Acceleration (PGA) due to the EQ-1, are equal to 0.087 and 0.156 g in the first case (11.46 s) and the second case (23.52 s), respectively, the numbers of PGA due to the EQ-2, are 0.09 and 0.117 g for the case 1 and the case 2, respectively. It is mentionable that the frequency content of the first is more than the second case, while the PGA, in the case 2 is occurred in the higher period. The acceleration response spectrum at the surface (5% damping) for the two cases is shown in Fig. 8. It illustrates that in the both of the earthquakes, the values of the spectral acceleration from 0.2 to 0.6 s belongs to the case 1, While after the 0.6 s, the peak response acceleration spectra, in the case 2, is almost by 2 times in the case 1. Figure 8 indicates that the highest acceleration is based on shear modulus degradation and damping curves computed using Eq. 5. Figure 9 reveals the amplification between the base motion and the surface. While, In the EQ-1, the amplification in the second case is about 2 times the first case (at low frequencies) but at the higher frequencies it is contrariwise. The number of amplification at the lower frequencies, in the EQ-2, is by 3 times in the EQ-1. Whereas, the value of amplification due to both of the two earthquakes in the soil deposit is almost identical.

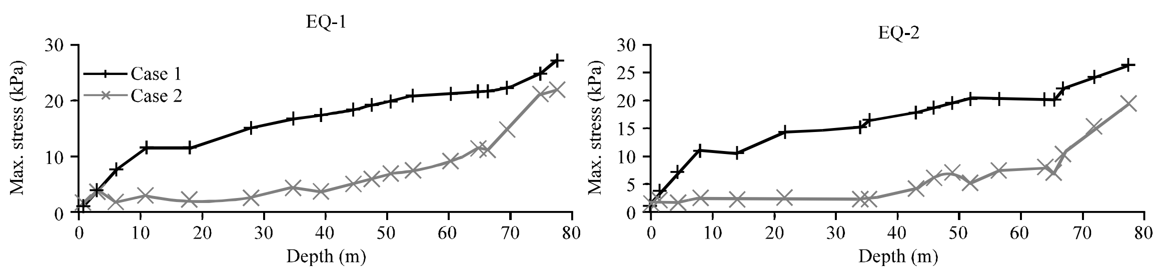

Fig. 5: The max. Stress in depth

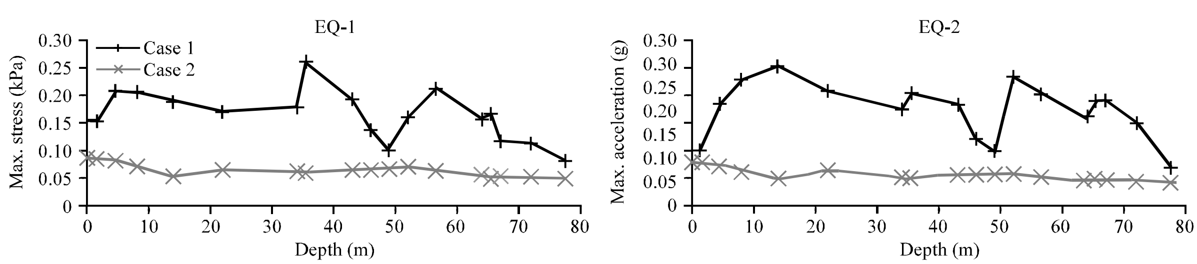

Fig. 6: The max. Acceleration in depth

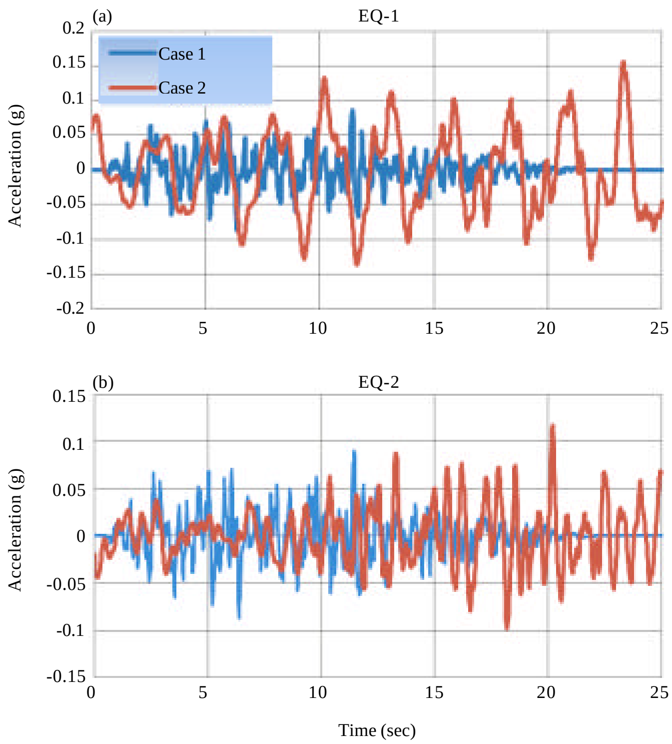

Fig. 7: Acceleration time histories at the surface

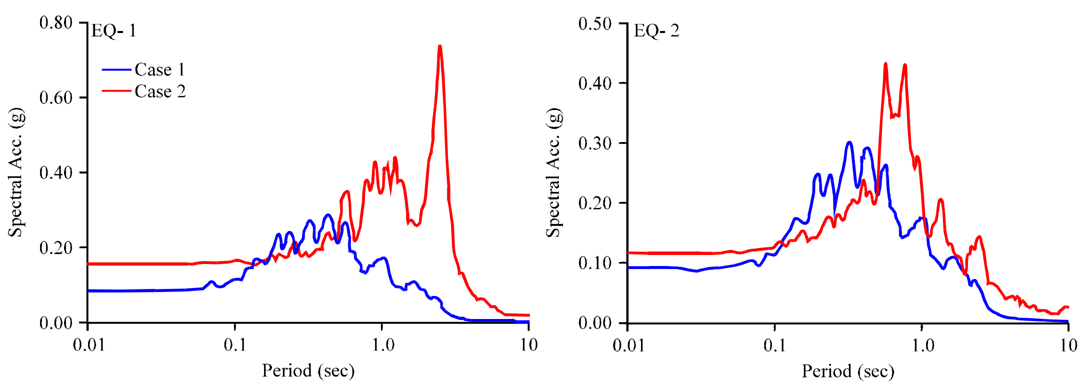

Fig. 8: Acceleration response spectra at the surface

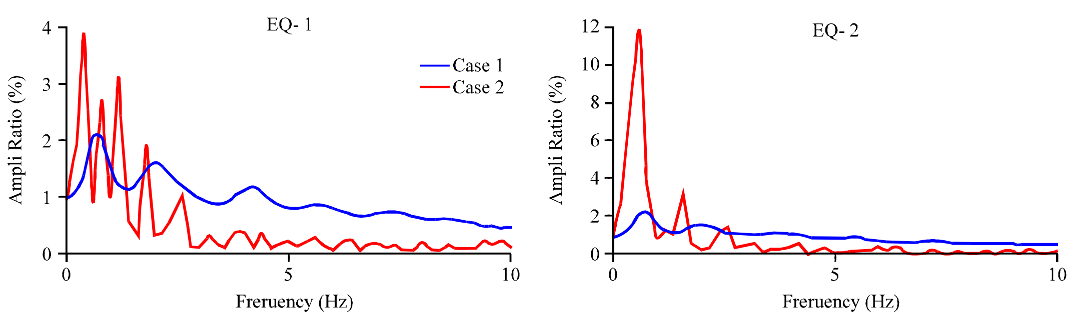

Fig. 9: Amplification between surface and base motion

CONCLUSION

The site response analysis affects the super structural design. Among the site response methods, the Equivalent Linear Method (ELM) is widely used due to simplification. In the ELM, the shear module and damping curves are very important. While, these curves have been developed by some researchers using one of the two factors: the effective confining pressure and the plasticity soil but some investigators have been presented several equations by the two effective factors in soil response analysis. Unified formulas were developed based on both of the two factors. To evaluate the influences of them, the soil profile of the PSC was simulated by the unified formulas and another model of the degradation curves (including one of the two factors). They were entered into the computer code developed based on the ELM. The analysis of results shows that the PGA values in the case 2 of the degradation curves are more than the case 1 model. When the confining pressure is varied in the curves, the damping of case 2 is less whereas the amplification becomes more. Also, the deep soil deposit significantly influences the effective confining pressure and the fundamental period. Although, the unified formulas are containing the two factors but the over consolidation ratio in the clayey soils is not considered.

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