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# Effects of Soil Model on Site Response Analyses

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## ABSTRACT

For the past two decades, earthquakes have demonstrated a change in the ground motion characteristics under the site conditions as the site analyses have taken an area of active studies. Non-linearity and equivalent linear methods are the two approaches popular and widely used in the design office. Based on the two methods, a number of computer codes have been developed. A deep soil deposit was analyzed and its properties were used as the input data required in the codes. The shear wave velocity was measured by empirical equation. The results of the two methods were compared. It was shown that the values of the acceleration and the strain determined by both equivalent linear and the nonlinear method were significantly greater.

**Key words:** Nonlinear site response, equivalent linear method, hysteresis loops, maximum shear modulus, effective damping ratio

## INTRODUCTION

Earthquake occurred, in the past two decades, have demonstrated that the site conditions play a prominent role in the earthquake characteristics at ground surface. Also, significant damages of structures supported by deep foundation in past earthquakes have developed to study of soil effects on the ground motions (Khari *et al*., 2013). This phenomenon is known as the site response analysis. Recently, the site response evaluation has taken an area of active research, so as the building and the bridge codes have been considered it. As the Fig. 1 illustrated, the response analysis is depend on soil geometry, mechanical behavior and base ground motion. The numerous investigators have been studied about the soil behavior during the earthquake and the influences of local geology and topography on the base ground motion as the input motion at the foundations (Motazedian *et al*., 2011; Khari and Bazyar, 2008). Based on the topography of the inter-layers boundaries, the three methods have been implemented for analyzing the site response: The one-dimension (1-D), the two-dimensions (2-D) and the three-dimension (3-D). Although, the earthquake waves are propagated in a 3-D continuous medium but their modeling is very complex. For this reason, the site response analyses is modeled with the adequately assumptions considered using the 2-D and the 1-D model. Between these two approaches, the one-dimensional approach is most frequently used for the site response analyses in practice. To compute the site analysis in the 1-D method, the soil shear strength is known as the most important factor. This factor is defined by the non-linearity of soil properties involving stiffness and damping of each identified soil layer.

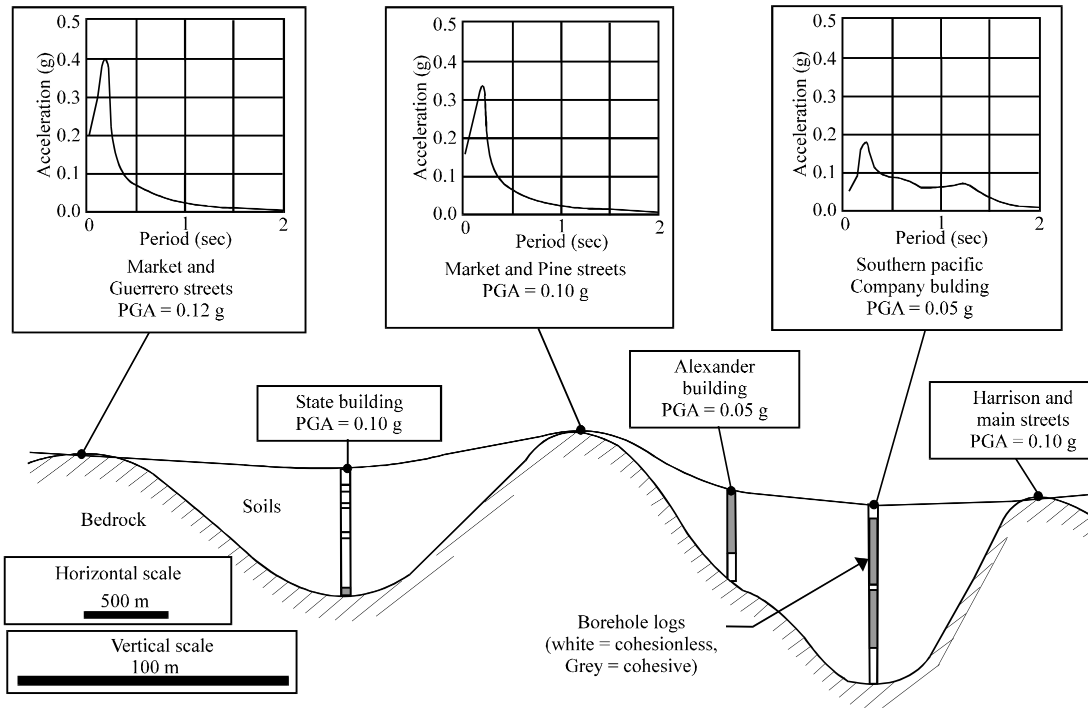


Fig. 1: San Francisco response for the 1957 Daly City Earthquake (Seed *et al.*, 1986)

So that, the non-linearity soil behavior causes the reduction of the soil shear strength during an earthquake (Khari *et al.*, 2011). Non-Linear Method (NLM) and Equivalent Linear Method (ELM) can usually model the non-linearity soil behavior. Meanwhile, using each of the two methods, the site response analysis can be evaluated but their influences on the acceleration occurred through the bedrock to ground surface are not similar. Based on the two methods, several computer codes have been developed such as SHAKE (Schanbel *et al.*, 1972) and DESRA (Lee and Finn, 1978).

This study summarized a series of the results attained from the site analysis using the non-linear and the equivalent linear method. A set of the soil data was entered into the two computer codes which were developed based on the above two approaches. The main objective of this paper is verifying the changes of the earthquake characteristics at the ground surface using the two methods.

## NON-LINEAR AND EQUIVALENT LINEAR METHOD

Vertically propagation of the shear waves through the uniform horizontally soil layers of the infinite lateral extent is the main assumption in the site response analyses. However, some assumptions such as compression waves and spatial variation of the ground motions are not taken into account conventionally.

The following equation is the most realistic model for site response analysis:

$$\begin{bmatrix} M_S & M_{SW} \\ M_{SW}^T & M_W \end{bmatrix}\begin{bmatrix} \bar{U} \\ \bar{W} \end{bmatrix}+\begin{bmatrix} 0 & 0 \\ 0 & H \end{bmatrix}\begin{bmatrix} \dot{U} \\ \dot{W} \end{bmatrix}+\begin{bmatrix} K_S & K_{SW} \\ K_{SW}^T & K_W \end{bmatrix}\begin{bmatrix} M_S^T \\ 0 \end{bmatrix}\bar{U}_g \tag{1}$$

where, $M_S$ and $M_W$ represent lumped masses of the soil skeleton and pore fluid, respectively; $K_S$ and $K_W$ denote the stiffness of the soil skeleton and pore fluid. $K^T_{SW}$ and $M^T_{SW}$ relate stiffness and mass coupling between solid and liquid phases. H is the dissipation matrix. However, a simple equation is to evaluate the site analyses as follows:

$$MU+CY+KU = MU_g \tag{2}$$

where, K, C and M are the stiffness, the viscous damping and the mass matrices of the soil deposit. On the other hand, the soil shear strength, during earthquake, is reduced owing to the vertically propagation of the shear waves. For considering this parameter, the soil can be modeled with three approaches: Viscoelastic (Kelvin-Voight), equivalent linear and hysteretic (nonlinear) approach. The main difference between equivalent linear and non-linear method is how the soil behavior subjected to cyclic loads is modeled. In fact, the equivalent linear model is the modified version of the Kelvin-Voight . In the Kelvin-Voight approach, the soil is simulated with spring and dashpot, as follows:

$$\tau = G\gamma+\eta\gamma \tag{3}$$

where, $\tau$, G and $\xi$ are the shear stress, the shear modulus and the damping, respectively; $\gamma$ and $\gamma'$ are the shear strain and its rate. At the beginning of the analyses, initial shear modulus and damping ratio are assumed. The maximum shear modulus ($G_{max}$) and the corresponding damping ratio are taken into account as the initial values. The soil stiffness and the damping are determined over the entire sequence of cyclic loads ($\gamma_{eff}$ = 0.65$\gamma_{max}$). The shear modulus degradation and damping ratio curves are described in the Fig. 2. It is remarkable that the equivalent linear method can be true for the shaking levels less than 0.4 g.

In the non-linear method, the soil behavior is evaluated by following the hysteresis loops (hysteretic model) with the Masing roles (Masing, 1926) and numerical integration in the time

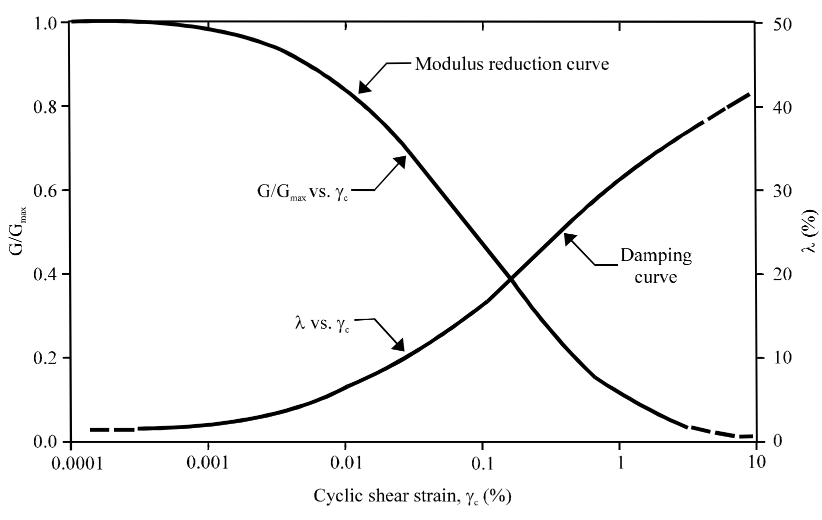


Fig. 2: Shear modulus degradation and damping ratio curves

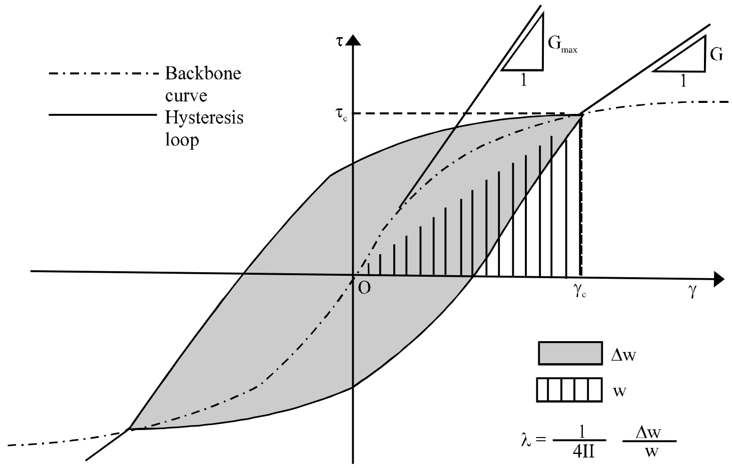


Fig. 3: Dynamic soil properties

domain. To take into account the non-linearity soil behavior, the stress-strain curves are modeled with a set of the mechanical elements (Iwan, 1967; Mroz, 1967). As earlier mentioned, the nonlinearity behavior can be illustrated by backbone curves. As Fig. 3 shown, the backbone curves can be formed using the hyperbolic model (Hardin and Drnevich, 1972) and depends on the nonlinearity parameters of soil. Therefore, the site response is sensitive to the input soil parameters. These parameters are the shear wave velocity, modulus reduction and damping curves, unit weight and peak shear strength. They can be measured in laboratory and field.

## CASE STUDY-SOIL CHARACTERISTICS

Case study studied was selected from Malaysia. The soil of Malaysia is clay and sandy-clay, mostly. The soil properties were simulated with the soil of the Penang Island and the results of the test done by Soil Center Lab SND. BHD. The depth deposit of this region is more than 100 m. While the shear wave velocity, in this depth is less than 500 m sec$^{-1}$ but, the result of the standard penetration testing blow count, N, is more than 50. To determine the shear wave Velocity (Vs), the empirical formula was used (Table 1) but Vs was not similar the soil type defined by some codes. As Fig. 4 shows the maximum shear wave velocity is equal to 320 m sec$^{-1}$. It is less than 700 m sec$^{-1}$ and as a result the component bedrock was not reachable. In selecting the time histories as the input motion, there are several factors such as earthquake magnitude, focal path, frequency content, site geology and duration. However, the relative importance of these factors varies in each case. Based on this information, the two ground motions were selected as input motion at the bedrock. As shown in Fig. 5, the maximum accelerations in the two earthquakes were 0.13, 0.22 g. The epicenter of the earthquakes were more than 700 (km) for the vertically propagated wave assumption. The results of the site response analyses were calculated by two computer codes developed based on the non-linear and the equivalent-linear method.

The damping ratios of 2 and 5% were assumed for the equivalent-linear and non-linear method, respectively. The degradation and damping curves have been developed based on the confining pressure by some researchers such as Seed *et al.* (1986), Seed and Idriss (1970), Sun *et al.* (1988) and Vucetic and Dobry (1991). Recently, the effect of plasticity index of soil was considered in these curves (Khari *et al.* 2011). In this study, normalized shear modulus degradation and damping ratio curves developed by Seed and Idriss (1970) were used for sand and clay.

Table 1: Soil profile in BH-9

| Soil | H (m) | ρ (KN m$^{-3}$) | $V_s$(m sec$^{-1}$) |
|---|---|---|---|
| Clay | 33.0 | 14.6 | 145 |
| Sand | 1.5 | 15.0 | 180 |
| Sand | 7.5 | 15.0 | 180 |
| Clay | 3.0 | 15.1 | 180 |
| Sand | 3.0 | 15.2 | 225 |
| Clay | 3.0 | 15.4 | 260 |
| Sand | 4.5 | 15.5 | 260 |
| Clay | 8.0 | 17.0 | 280 |
| Sand | 1.5 | 17.2 | 295 |
| Clay | 10.0 | 18.5 | 320 |

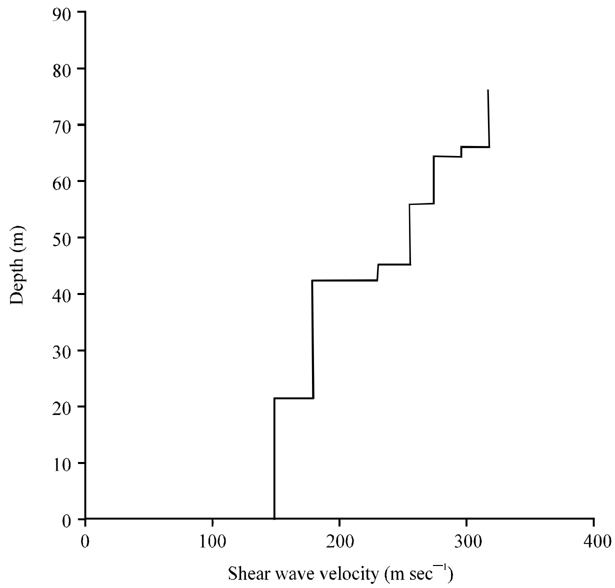


Fig. 4: Shear wave velocity profile

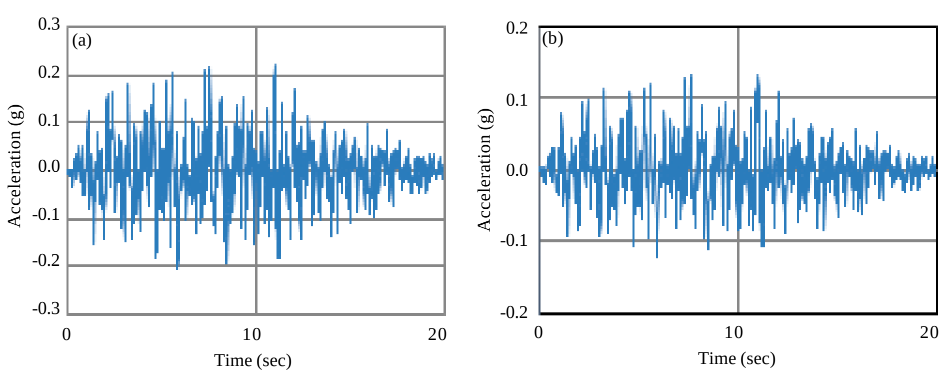


Fig. 5: Acceleration time histories

## DISCUSSION

Figure 6 shows the maximum stress versus depth. As it is recognized, the maximum stress computed, in the ELM method, is more than that of the NLM method. It can be due to the used

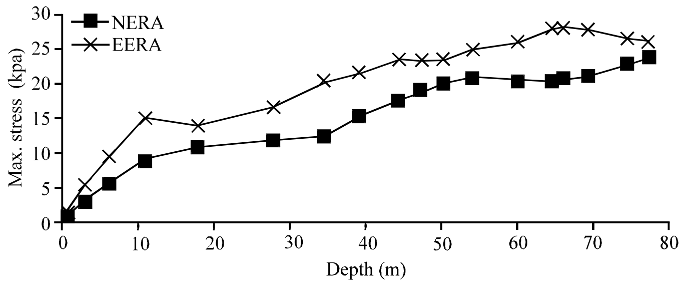


Fig. 6: Max. stress in the depth

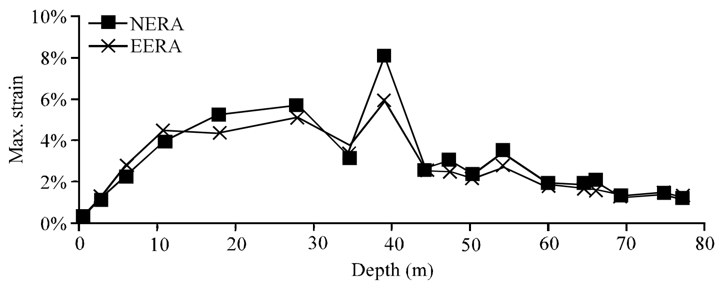


Fig. 7: Max. strain in the depth

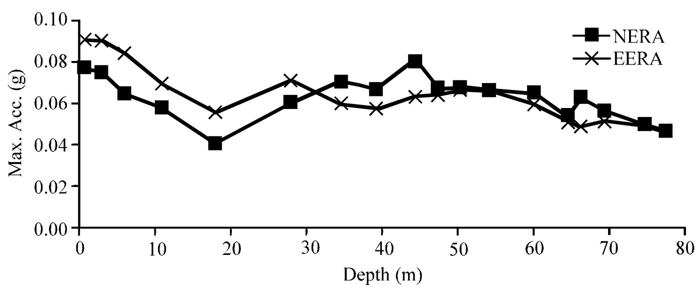


Fig. 8: Max. acceleration in the depth

strain level of the soil deposit in each term of the calculation. While, this amount is about 25 kpa in the end of soil profile using two methods but it decreases to zero at the surface. This can be in a good agreement with the fundamental concepts of the site effects on the time histories at the ground surface as shown by Lee and Finn (1978) using DERSA2 code. The values of the maximum strain are similar in the two methods only in the middle of soil profile that these amounts, in the ELM, are more than the NLM (Fig. 7) similar to the results obtained by Phillips and Hashash (2009) although Rayleigh Damping was utilized in their simulation. This type of the damping need the two mode of the excitation for a determination. Based on these results, it is mentionable that the viscous damping can be more acceptable in the foregoing two methods. This inconsistency can be affected by the damping ratio curves. As Fig. 8 exhibits the maximum accelerations are almost equal from the middle to the end of soil profile but they are not equal at the ground surface, so that the maximum acceleration in the ELM, is more than its amount in the NLM. This amplification has demonstrated but its coefficients were different in the frequency and the time domain analysis.

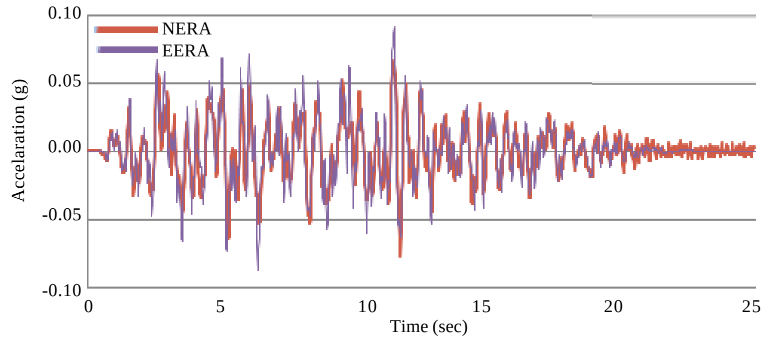


Fig. 9: Acceleration time histories at surface

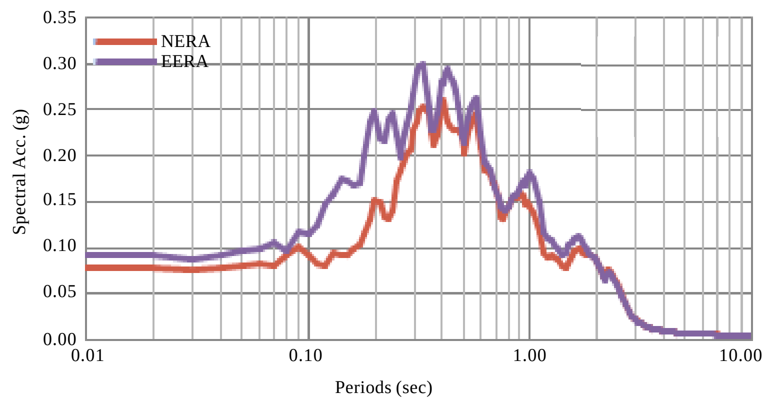


Fig. 10: Acceleration response spectra

Moreover, the peak acceleration in the input motion affects this coefficient owing to the variation of the strain level in the each depth and consequently the soil stiffness degradation during the seismic excitation. it can be demonstrated based on the experimental tests performed by Wilson *et al*. (1997). The two accelerograms obtained are almost consistent in the two methods (Fig. 9) but the maximum accelerations are 0.077 g and 0.09 g in the non linear method and the equivalent linear method, respectively which is corroborated by Fig. 8. This difference may be due to the effective strain level ($\gamma_{eff} = 0.65\gamma_{max}$) because in the nonlinear method the really strain is computed for the next step. Further, when the soil behavior is simulated using the Kelvin-Voight mechanical element in the equivalent method, the soil properties are equaled during a loading and unloading process. However, Lopez-Caballero *et al*. (2006) suggested that the soil should be modeled using the elastoplasticity theory because the soil properties vary with different strain levels. The response spectra are presented in Fig. 10. It explains that in the equivalent linear method, the acceleration response spectrum is in remarkable agreement with the nonlinear method, especially in the higher periods. It is reemphasized the earthquake characteristics are modified under the site conditions and these modifications are depend on the analyses method.

## CONCLUSION

To evaluate the site response analyses under the earthquake loading, the two approaches have been developed: The equivalent linear and the nonlinear method. The dynamic properties of the soil deposit are considered in the computation by the two methods so that how their modeling is known as the main difference between the two approaches. Based on these methods, a number of computer codes have been developed. EERA (Equivalent Earthquake site Response Analyses) and NERA (Nonlinear Earthquake site Response Analyses) are developed. Data input required in the two programs were included of the dynamic and the general properties of each identified soil layer. The most important dynamic parameter was the shear wave velocity that which was measured empirically. Also, a series of laboratory tests were done to estimate the unit weight of the soil and its plasticity index. Two ground motions were selected as input motion. They occurred in the Sumatra Island in Indonesia. After analyzing and comparing the results obtained, it was recognized that the maximum stress values computed by the ELM approach were higher. On the contrary, the maximum strain values in the nonlinear method were greater. It is worthy of note that the maximum acceleration values computed by the two methods were replaced at the middle of soil profile. In addition, the maximum acceleration calculated at the ground surface with the equivalent approach was greater than with the nonlinear method. Numerical results achieved by the two approaches have been shown that the values computed by the equivalent linear method are significantly more even in compared with the acceleration scaled. Consequently, it can not capture the nonlinearity of soil behavior well.

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