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Effect of Loading Frequency Dependent Soil Behavior on Seismic Site Effects

하중의 주파수에 의하여 지배받은 흙의 동적 거동이 부지증폭현상에 미치는 영향

Park, Duhee[1] 박 두 희 Hashash, Y. M.A.[2] 하샤시 유세프

Lee, Hyun-Woo[3] 이 현 우 Kim, Jae-Yoen[3] 김 재 연

요 지

등가선형해석은 지반증폭현상을 모사하기 위하여 널리 사용되고 있으며, 해석 시 흙의 거동은 하중의 주파수의 영향을 받지 않는다고 가정되어왔다. 반면, 실내시험은 점성토의 경우 하중의 주파수의 영향을 크게 받는다는 것을 보여주고 있다. 본 연구에서는 하중의 주파수가 흙의 동적 거동에 미치는 영향을 고려하는 새로운 등가선형해석기법이 개발되었으며 주파수의 영향을 규명하기 위하여 지반응답해석을 수행하였다. 해석 결과, 하중의 주파수에 따라서 변화하는 전단탄성계수가 지반응답에 미치는 영향은 작은 반면 감쇠비는 큰 영향을 끼치는 것으로 판명되었다. 이는 하중의 주파수가 높아질수록 흙의 감쇠비도 같이 증가하며 이로 인하여 지진파의 고주파수 요소가 필터링 되기 때문이다. 따라서, 하중의 주파수에 지배 받는 흙의 거동은 특히 고주파수 요소가 풍부한 지진파 전파 모사 시 특히 중요하다고 판단된다.

Abstract

Equivalent linear analysis is widely used in estimating local seismic site effects. The soil behavior in the analysis is often assumed to be rate-independent and is not influenced by the seismic loading frequency. Laboratory results, however, indicate that cohesive soil behavior is greatly influenced by the loading frequency. A new equivalent linear analysis method that accounts for the loading frequency dependent soil behavior is developed and used to perform a series of one dimensional site response analyses. Results indicate that while frequency dependent shear modulus has limited influence on computed site response, frequency dependent soil damping greatly filters out high frequency components of the ground motion and thus results in lower response.

Keywords : Damping, Equivalent linear, Loading frequency, Shear modulus, Site response analysis

1. Introduction

Earthquake activities, including strong motion records from recent earthquakes including the 1989 Loma Prieta, 1994 Northridge, 1995 Hyogoken-Nanbu, and 1999 Chi-Chi events, have shown the importance of local site conditions on propagated ground motions. One-dimensional (1-D) site response analysis is widely performed to account

1 Member, Full-time Lecturer, Dept. of Civil Engrg., Hanyang Univ., dpark@hanyang.ac.kr
2 Associate Prof., Dept. of Civil & Environmental Engrg., Univ. of Illinois at Urbana-Champaign
3 Member, Graduate Student., Dept. of Civil Engrg., Hanyang Univ.

for local site effects during an earthquake (Idriss, 1990; Kramer, 1996; Hashash and Park, 2001). Vertically propagating horizontal shear waves (SH waves) approximate the ground motion, and horizontal soil layers represent site stratigraphy. Soil behavior is often approximated as a Kelvin-Voigt solid with a linear elastic shear modulus and viscous damping. Solution of wave propagation equation is performed in either frequency or time domain.

For strong vibrations (medium and large earthquakes), linear elastic solution is no longer valid since soil behavior is inelastic, non-linear and strain dependent. Equivalent linear analysis, performed in frequency domain, has been developed to approximate the nonlinear behavior of soil (Schnabel et al., 1972). The equivalent linear method approximates nonlinear behavior by incorporating shear strain dependent shear modulus and damping soil curves. However, a constant linear shear modulus and damping at a representative level of strain are used throughout the analysis, contrary to the nonlinear analysis method in which the dynamic equation of motion is integrated in time domain and the nonlinear soil behavior is accurately modeled. Although the equivalent linear analysis is an approximate method, it has proven to be very effective in modeling ground motion propagation (Idriss, 1990).

Laboratory tests show that the soil behavior is dependent on the rate of loading, in which both the shear modulus and damping ratio changes with the frequency of the loading. In both equivalent linear and nonlinear analysis, such behavior has been ignored for the soil behavior is assumed to be rate independent. There is yet no available method that can account for the rate dependency of soil behavior and its effect on site response analysis. This paper develops a new equivalent linear analysis procedure that accounts for the rate-dependency and determines its effect on ground motion propagation.

2. Laboratory Testing of Loading Frequency Dependent Soil Behavior

Cyclic soil behavior is commonly represented by shear modulus reduction and damping curves, as shown in Fig. 1. Several factors are known to influence cyclic soil behavior, which include effective confining pressure (EPRI, 1993; Laird and Stokoe, 1993), plasticity index (Vucetic and Dobry, 1991), and rate (or frequency) of loading (Kramer, 1996; Isenhower and Stokoe, 1981). While the effect of confining pressure and plasticity index is well documented, the effect of rate of loading is recognized but not as well quantified.

Earliest studies on the effect of rate of loading on soil stiffness include laboratory tests performed by Whitman (1957), Richardson and Whitman (1963), Yong and Japp (1967), Olson and Parola (1967). The results of the tests indicate that the stiffness increases with increase in rate of shearing. The reason for this phenomenon is attributed to the viscous nature of soils. As the rate of loading and shearing decreases, more time is allowed for the soil to creep and relax, therefore resulting in larger deformations and lower shear stress (Matesic and Vucetic 2003). The studies also show dependence of the influence of rate of loading on the plasticity index of soils, and hence clays showing more dependence on rate of loading compared to sands.

Isenhower and Stokoe (1981) used resonant column (RC) /torsional shear (TS) apparatus to investigate the effect of rate of loading on shear modulus using San Francisco Bay Mud. Fig. 2 shows the variation of shear modulus with shear strain and average shear strain rate. The average shear strain rate is defined as

$$\dot{\gamma} = 4f\gamma \qquad (1)$$

where $\dot{\gamma}$ = average shear strain rate, f = loading frequency, and γ = shear strain amplitude. For a given shear

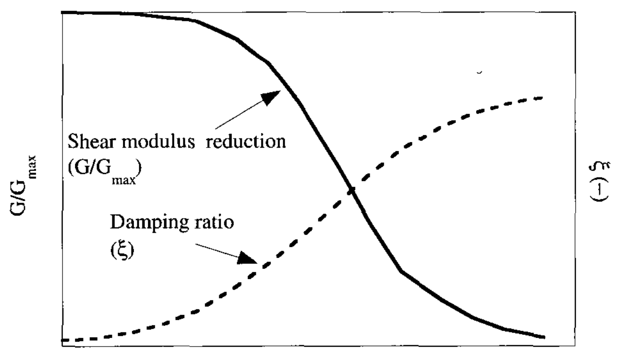

Fig. 1. Shear modulus reduction and damping ratio curves

strain amplitude, the rate of loading (shearing) will increase with increase in the loading frequency, f. The shear modulus increases with increase in frequency of the loading, displaying a linear relationship with lognormal of the frequency. Another characteristic of the tests is the dependence on shear strain amplitude, whereby the rate of increase in shear modulus with shear strain rate decreases at higher strain levels, except at shear strain amplitude of 0.0355%.

Kim et al. (1991) performed extensive laboratory tests to characterize the effect of rate of loading on shear modulus and damping ratio, using both RC and TS tests. TS tests are used for loading frequency up to 10 Hz and RS tests are used for higher loading frequencies. The loading frequencies used in the tests range from 0.05 to 85 Hz at strain amplitudes 0.001 and 0.1%. Various soil samples are used in the study, including dry sand, undisturbed cohesive soils, and compacted soil subgrades. The effect of loading frequency on dry sand was negligible, while the cohesive soil was heavily influenced by the rate of loading. Fig. 3a shows the influence of the loading frequency on shear modulus at strain amplitude of 0.001% and 0.01% using undisturbed cohesive soil obtained from Treasure Island at a depth of 18.3 m. The rate of increase in shear modulus with loading frequency is similar at both strain levels. To better characterize the influence of the loading frequency on shear modulus, the shear modulus was normalized to the shear modulus obtained at a loading frequency of 0.5 Hz, as shown in Fig. 3b. At both strain levels, the dependence on loading frequency is almost identical.

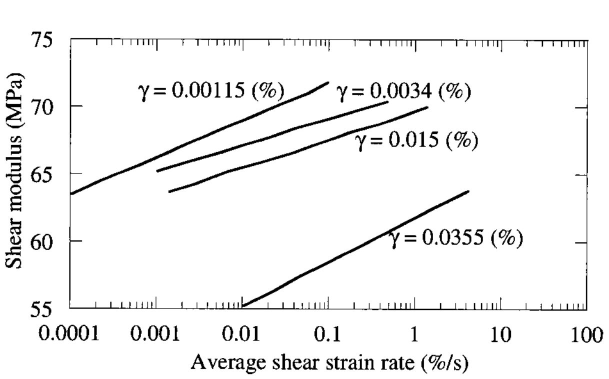

Fig. 2. Effect of average strain rate on secant shear modulus (modified after Isenhower and Stokoe, 1981)

Matesic and Vucetic (2003) studied the effects of the shear strain rate $\dot{\gamma}$ on secant shear modulus of three clays and three sands at small cyclic shear strain amplitudes ranging between 0.0003 and 0.02% and $\dot{\gamma}$ between 0.0002 and 0.04%/s. For all six soils shear modulus increases with shear strain rate, such that the shear modulus versus log $\dot{\gamma}$ data plots approximately along a straight line. Fig. 4 shows the test results using the kaolinite. Fig. 4a shows the shear modulus versus average shear strain rate, Fig. 4b shows the shear modulus versus loading frequency, and Fig. 4c shows the normalized shear modulus. The normalized shear moduli are not much influenced by the imposed shear strain amplitudes, except at shear amplitude of 0.0005%. Compared with the results of (Kim et al. 1991), the normalized shear moduli are very similar, as shown in Fig. 4c.

To represent the complete spectrum of shear strain amplitudes that the soil experiences during a seismic loading, there is a need to define the dependence of the shear modulus on loading frequency at larger shear strains. Fig. 5 compares the shear modulus reductions curves obtained from the TS and RC test (Kim et al. 1991). The

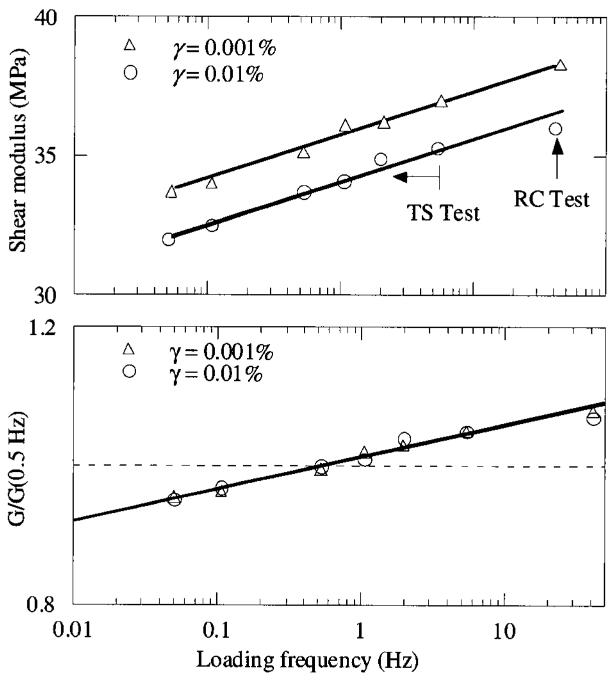

Fig. 3. Variation of (a) shear modulus and (b) normalized shear modulus of undisturbed cohesive soil with loading frequency (Kim et al., 1991). The shear modulus is normalized to the shear modulus measured at 0.5 Hz.

RC test results in higher shear modulus up to 0.01%, consistent with the laboratory tests presented previously. However, the discrepancy between the two tests quickly decays at higher strain amplitudes, converging at strain amplitude of 0.1%. One reason for the similarity between the two tests at large strain amplitude is the reduction in the frequency dependence of the shear modulus at high strain levels. Another reason is the cyclic degradation effect using the RC tests. It is thus difficult to identify the exact frequency dependency of the shear modulus at large strains and further research is needed. In this study, it is assumed that the cyclic degradation effect is negligible and the reduction in shear modulus is only due to the frequency dependence.

The loading frequency not only influences the shear modulus, but also has pronounced effect on the measured damping ratio. The variation in the damping ratio of undisturbed cohesive soils with loading frequency using TS and RC tests is shown in Fig. 6 (Kim et al., 1991). The damping ratio is again normalized to the damping ratio measured at 0.5 Hz. The grey lines represent the upper and lower bound while the black lines represent the average. The damping ratio is independent of frequency below the loading frequency of 1 Hz but increases at higher frequencies. When compared with the representative damping ratios at two shear strain amplitudes, the de-

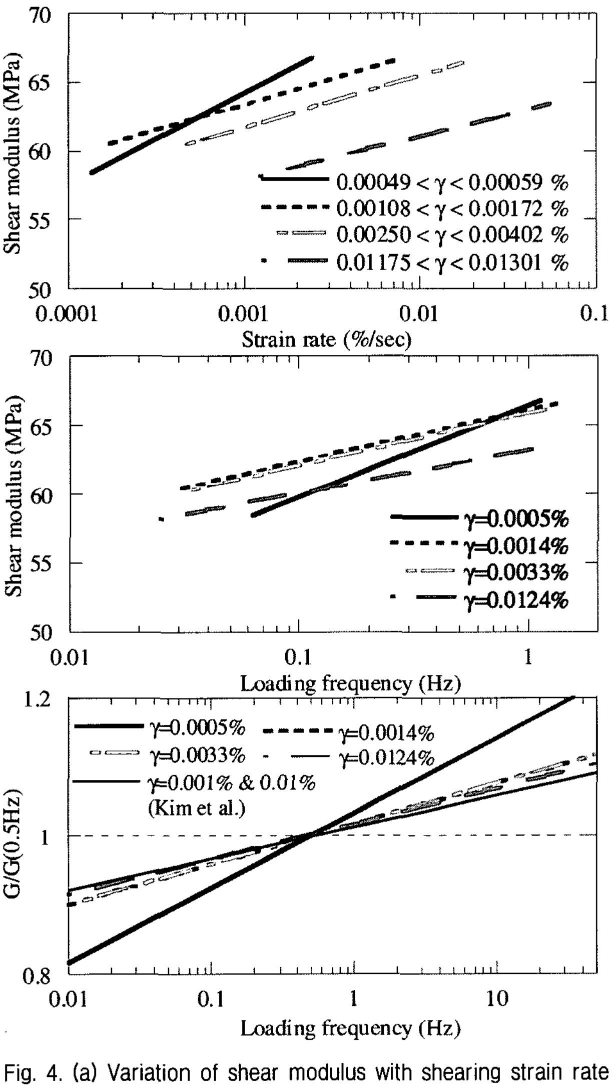

Fig. 4. (a) Variation of shear modulus with shearing strain rate (Matesic and Vucetic, 2003), (b) variation of shear modulus with loading frequency (based on data of Matesic and Vucetic, 2003), and (c) normalized shear modulus (based on data of Matesic and Vucetic, 2003)

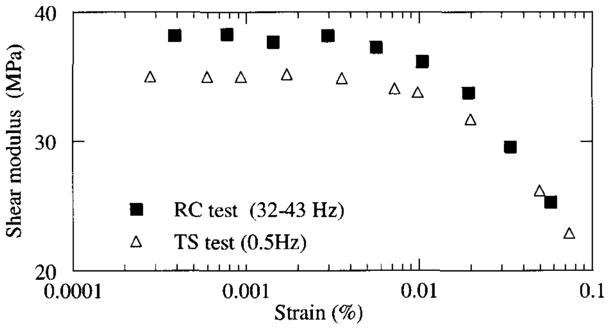

Fig. 5. Comparison of shear modulus reduction curves obtained from RC and TS tests (Kim et al., 1991)

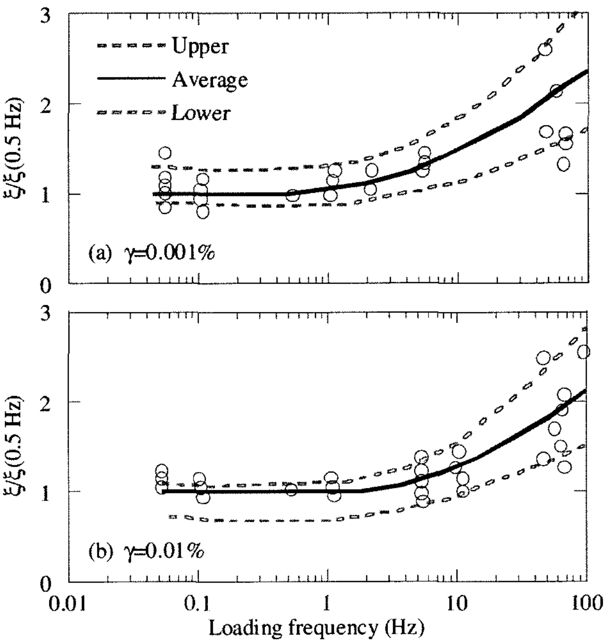

Fig. 6. Variation of normalized damping ratio with loading frequency based on RC and TS tests (Kim et al., 1991). Damping ratio is normalized to the damping ratio obtained at a loading frequency of 0.5 Hz.

pendence of the loading frequency decreases at higher shear strain amplitude, as shown in Fig. 6b.

Rix and Meng (2005) propose use of a non-resonance (NR) test to measure the frequency dependence of dynamic soil properties. The NR method uses frequency response function between the loading harmonic torque and resulting rotation of the specimen. The main advantage of the test apparatus is its ability to eliminate equipment-generated damping due to the interaction between solenoids and magnets within the motor. Several remolded and undisturbed soil specimens are used to characterize the effect of loading frequency on shear modulus and damping ratio. The tests are performed at shear strain amplitudes less than the linear threshold strain of the tested soil samples. Fig. 7a and b show that the damping has a parabolic shape, unlike the test results by Kim et al. (1991) which display constant damping at frequencies lower than 1 Hz. The trend at high frequencies is similar, whereby the shear modulus and the damping ratio increase with increase in frequency.

The discrepancy between the measured damping ratios of the two studies demonstrates that additional study on the dependence of the soil behavior on rate of loading is warranted to characterize the frequency dependent soil damping.

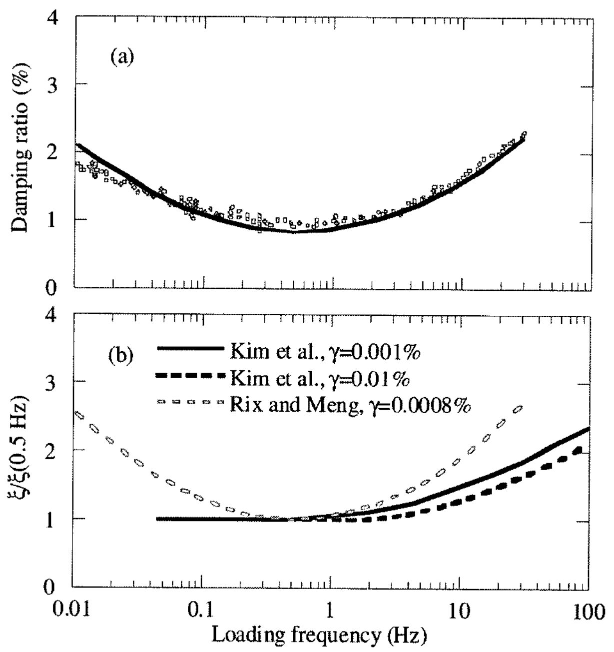

Fig. 7. Variation of damping ratio with loading frequency, (a) damping ratio using remolded kaolinite at shear strain amplitude of 0.0008% (Rix and Meng, 2005) (b) comparison of normalized damping ratio curves of Kim et al. (1991) and Rix and Meng (2005)

3. One Dimensional Equivalent Linear Solution

The 1-D equation of motion for vertically propagating shear waves through unbounded medium can be written as:

$$\rho\frac{\partial^2 u}{\partial z^2}=\frac{\partial \tau}{\partial z} \tag{2}$$

where ρ = density, τ = shear stress, u = displacement and z = depth below ground surface.

Soil behavior is approximated as a Kelvin-Voigt solid. The shear stress - shear strain relationship is expressed as:

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

where G = shear modulus, γ = shear strain and η = viscosity.

Substituting Eq. (3) into (2) results in:

$$\rho\frac{\partial^2 u}{\partial t^2}=G\frac{\partial^2 u}{\partial z^2}+\eta\frac{\partial^2 u}{\partial z^2 \partial t} \tag{4}$$

Eq. (4) can be solved for a harmonic wave propagating through a multi-layered soil column (Schnabel et al., 1972), as shown in Fig. 8. Introducing a local coordinate

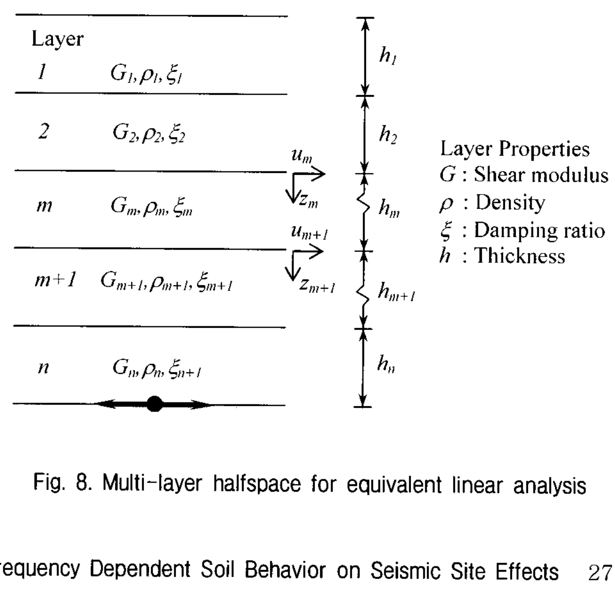

Fig. 8. Multi-layer halfspace for equivalent linear analysis

Z for each layer and solving the wave equation, the displacement at the top and bottom of each layer becomes:

$$u(Z_m = 0, t) = u_m = (A_m + B_m)e^{i\omega t}$$
$$u(Z_m = h_m; t) = u_{m+1} = (A_m e^{ik_m^* h_m} + B_m e^{-ik_m^* h_m})e^{i\omega t} \quad (5)$$

where u = displacement, A_m and B_m = amplitudes of waves traveling upwards (-z) and downwards (z), h_m = thickness, and $k_m^* = \frac{\omega}{(\nu_s)_m(1+i\xi_m)}$ for each layer. Applying the boundary conditions and compatibility requirements will result in the recursive formulae for successive layers:

$$A_{m+1} = \frac{1}{2}A_m(1+\alpha_m^*)e^{ik_m^* h_m} + \frac{1}{2}B_m(1-\alpha_m^*)e^{-ik_m^* h_m}$$
$$B_{m+1} = \frac{1}{2}A_m(1-\alpha_m^*)e^{ik_m^* h_m} + \frac{1}{2}B_m(1+\alpha_m^*)e^{-ik_m^* h_m} \quad (6)$$

where $\alpha_{m+1}^* = \frac{\rho_m(\nu_s)_m(1+i\xi_m)}{\rho_{m+1}(\nu_s)_{m+1}(1+i\xi_{m+1})}$.

The motion at any layer can be easily computed from motion at any other layer (e.g. input motion imposed at the bottom of the soil column) using the transfer function that relates displacement amplitude at layer i to that at layer j:

$$F_{ij}(\omega) = \frac{|u_i|}{|u_j|} = \frac{A_i(\omega)+B_i(\omega)}{A_j(\omega)+B_j(\omega)} \quad (7)$$

The transfer function of the ratio of the Fourier spectrum of the motion at layer i is equivalent to that at layer j. k_m^* in Eq. (3) and α_m^* in Eq. (4) implies that the shear modulus and damping property are independent of loading frequency, ω.

The frequency domain solution is predicated on the assumption that modulus and damping properties are constant and independent of the strain level. The equivalent linear approximation method was developed to capture non-linear cyclic response of soil within the framework of the frequency domain solution (Schnabel et al., 1972). The nonlinear hysteretic stress-strain behavior is approximated by the modulus degradation and damping curves. For a given ground motion time history, the propagated ground motion is calculated using an initial estimate of modulus and damping values. The computation is performed in the frequency domain. Then, the strain time histories for each layer, from which the maximum strain values are obtained, are calculated. An effective shear strain (equal to about 65% of peak strain) is computed for a given soil layer and corresponding estimates of shear modulus and damping are obtained from the shear modulus reduction and damping curves. This process is repeated until a converged solution is reached.

4. Development of New Equivalent Linear Procedure that Accounts for the Loading Frequency Dependent Soil Behavior

A new equivalent linear analysis procedure that can account for the frequency dependent soil behavior is developed. The new procedure is built upon the currently available one-dimensional site response code DEEPSOIL (Hashash and Park, 2001; Hashash and Park, 2002; Park and Hashash, 2004). DEEPSOIL allows performing both non-linear and equivalent linear site response analysis. The equivalent linear analysis feature is very similar to SHAKE (Schnabel et al. 1972), with no limitation on the number of layers and material properties. In addition, various types of complex shear moduli can be selected (Park, 2004).

New procedure development is composed of two steps. The first step defines the loading frequency dependent shear modulus and damping ratio. The second step modifies the transfer function to account for the frequency dependent shear modulus and damping ratio.

4.1 Characterization of Loading Frequency Dependent Shear Modulus and Damping Ratio

The first step is characterizing frequency dependent shear modulus and damping curves based on the laboratory results, shown in Fig. 2 to Fig. 7. The frequency dependent shear modulus will be first characterized, followed by the damping ratio.

Firstly, a shear modulus reduction curve that is representative of the dynamic soil behavior is selected. The curve will be termed reference modulus reduction curve. When defining the soil curve, the frequency at which the reference curve was obtained should be known. Such frequency will be termed the reference frequency.

The effect of the frequency on shear modulus is defined with respect to the reference curve. The procedure utilizes normalized functions that relate the shear modulus at a given frequency with the reference shear modulus (Fig. 9a). The functions, defined at shear levels 0.001%, 0.01%, and 0.1%, respectively, describe ratio of the shear modulus at various loading frequencies to the reference shear modulus. Such ratio will termed Gratio in this study. Note that the functions are normalized to the reference frequency.

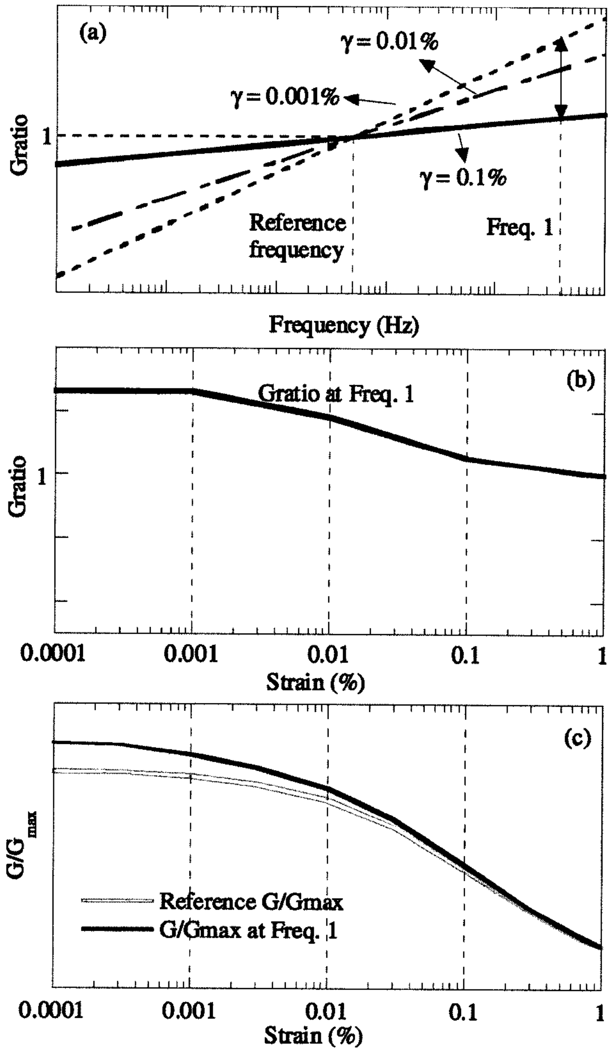

Fig. 9. Frequency dependent shear modulus characterization: (a) Gratio vs. frequency, (b) Gratio vs. strain at the selected frequency Freq. 1, (c) comparison of reference shear modulus and G/Gmax at Freq. 1

The shear modulus at any frequency and strain level can be calculated by multiplying the Gratio from the function to the reference shear modulus. If the loading frequency is equivalent to the reference frequency, Gratio is 1 independent of the shear strain level. At other loading frequencies, the rate of dependency is obtained from the Gratio functions. Fig. 9 demonstrates how a frequency dependent shear modulus reduction curve is constructed at Freq. 1, which is higher than the reference frequency. The normalized functions are used to determine Gratios at three strain levels (Fig. 9a). It is assumed that the Gratio varies linearly between the defined shear strain levels and is constant at stain levels lower than 0.001%, and Gratio is assumed to be 1 at strain level = 1.0% (Fig. 9b). The shear modulus reduction curve at Freq. 1 is shown in Fig. 9c. Since increase in frequency results in increase in shear modulus, the resulting shear modulus lies above the reference shear modulus reduction curve. At a frequency lower than the reference frequency, the shear modulus reduction curve will be lower than the reference shear modulus.

The same procedure is applied in developing frequency dependent damping curves. The procedure applies the ratio of the damping at a given frequency to the damping ratio at the reference frequency, termed as the Dratio. Fig. 10 demonstrates how a frequency dependent damping ratio is constructed at Freq. 1, which is higher than the reference frequency. Only two sets of normalized functions, defined at shear strain amplitudes of 0.001% and 0.01%, are used in the example. DEEPSOIL is capable of defining up to frequency dependent functions at three strain levels. However, only two curves are used due to the lack of data at higher strains. Note that the damping ratio curves do not converge at strains higher than 0.1% (Fig. 10c), as in shear modulus reduction curves. This is because the damping ratios are combination of viscous damping and hysteretic damping. While the influence of the loading frequency on hysteretic damping will be very small at high strain levels, the influence on the viscous damping will still remain as before.

4.2 Development of Frequency Dependent Transfer Function

The equivalent linear analysis defines the transfer function at each frequency. Constant values of shear modulus and damping ratio at a representative shear strain amplitude independent of loading frequency are assigned in defining k^*_m and α^*_m, of Eq. (5) and Eq. (6), which are used to calculate the transfer function, Eq. (7). The transfer function is modified as follows to account for the frequency dependent soil behavior by modifying k^*_m and α^*_m:

$$k^*_m = \frac{\omega}{\left[\sqrt{G_m(\omega)/\rho_m}\,\right]_m [1+i\xi_m(\omega)]} \tag{8}$$

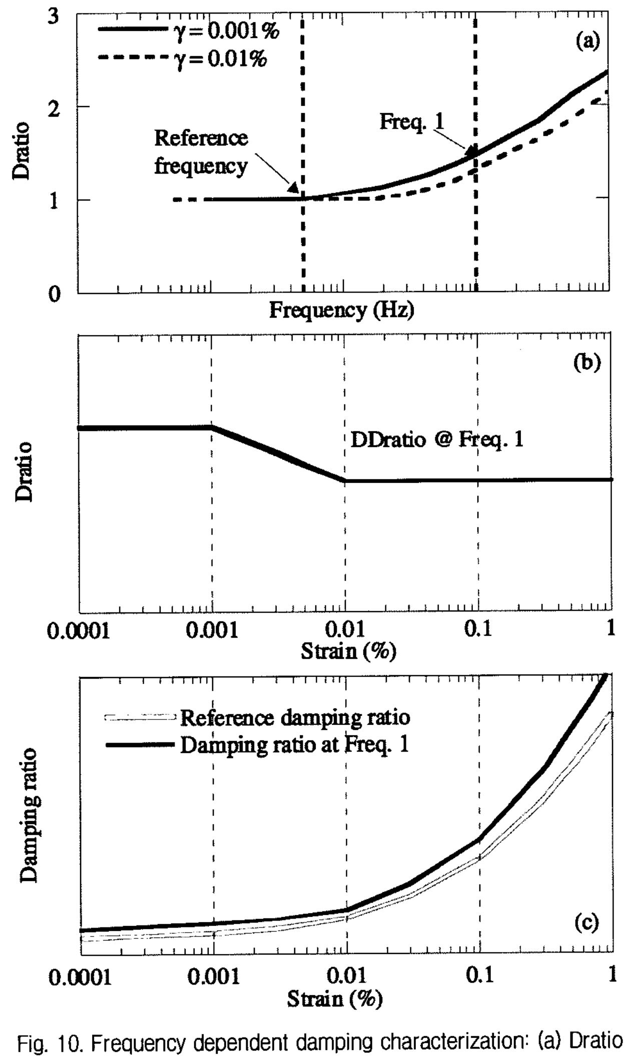

Fig. 10. Frequency dependent damping characterization: (a) Dratio vs. frequency, (b) Dratio vs. strain at the selected frequency Freq. 1, (c) comparison of reference damping ratio and damping ratio at Freq. 1

$$\alpha^*_{m+1} = \frac{\rho_m\left[\sqrt{G_m(\omega)/\rho_m}\,\right]_m [1+i\xi_m(\omega)]}{\rho_{m+1}\left[\sqrt{G_m(\omega)/\rho_m}\,\right]_{m+1} [1+i\xi_{m+1}(\omega)]} \tag{9}$$

By using frequency dependent shear modulus and damping ratio in defining k^*_m and α^*_m, the transfer function becomes frequency dependent. This function is newly incorporated in DEEPSOIL. Note that the developed procedure does not account for the non-linear behavior of soil, since it is built upon the equivalent linear analysis. Thus, the rate dependency is defined at the representative strain calculated for each layer.

5. Influence of Frequency Dependent Soil Behavior on Site Response Analysis

A series of equivalent linear analyses are performed to characterize the effect of the frequency dependent soil behavior on site response analysis. The analyses use a range of idealized soil profiles and soil properties. Two soil columns, 50 and 300 m thick, are used in the analysis, as shown in Fig. 11. Two constant shear wave velocity profiles, 200 m/sec and 500 m/sec respectively, with homogeneous soil properties are used. The shear wave velocity of the bedrock is 760 m/sec. Fig. 11b shows the shear modulus reduction curve and damping ratio curve used in the analysis. The curves, developed by Vucetic

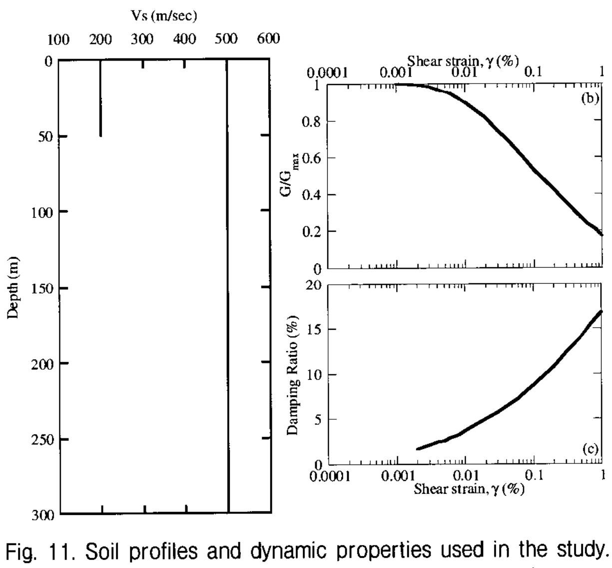

Fig. 11. Soil profiles and dynamic properties used in the study. The shear wave velocity of the bedrock is 760m/sec. The dynamic curves are representative of cohesive soils having PI = 30 (Vucetic and Dobry, 1991).

and Dobry (1991), are selected to represent cohesive soils with plasticity index of 30. It is assumed that the selected curves are developed at a reference frequency of 0.5 Hz.

The results of (Kim et al., 1991) are used to define the frequency dependent shear modulus, as shown in Fig. 3. Identical Gratio is assumed for shear strain amplitudes of γ = 0.001% and γ = 0.01%, due to the similarity of the ratios. Gratio is set to 1 at strains higher than γ = 0.1%.

Three sets of frequency dependent damping ratio curves are used in the analysis. First set, referred to as Type 1, is based on the results of Rix and Meng (2005). When using this set, it is assumed that the frequency dependence is identical at all strain levels, since the behavior at higher strains is not known. The second set, referred to as Type 2, is identical to Type 1 except at low frequencies at which Dratio is set to 1. Type 2 is defined to investigate the effect of high Dratio at low frequencies. The third set, referred to as Type 3, is based on the results of Kim et al. (1991). Two different damping ratios are defined at strain amplitudes of γ = 0.001% and γ = 0.01%, as shown in Fig. 7. At higher strains, it is assumed that the frequency dependence is identical to the dependence at γ = 0.01%.

Two types of motions are used, as shown in Fig. 12. One motion is the synthetic motion generated by SMSIM (Boore, 2000), using input parameters used in developing the probabilistic seismic hazard maps of the United States (Frankel, 1996). The motion is representative of ground motion from an earthquake with magnitude (M) = 7 and epicentral distance (R) = 30 km. The peak ground acceleration of the motion is 0.63 g. The second motion used is the recorded motion at Yerba Buena Island during the Loma Prieta earthquake, 1989. The peak ground acceleration of the motion is 0.067 g. Compared with the synthetic motion, the recorded motion has significantly less high frequency components.

Fig. 13 compares computed surface response spectra of 50 m thick, Vs = 200 m/sec profile using frequency independent (FI) and frequency dependent (FD) soil properties. The input motion is the synthetic motion. Three analyses are performed, using a) FI shear modulus and FI damping, b) FD shear modulus and FD damping (Type 1), and c) FI shear modulus and FD damping (Type 1). Surface response spectrum using FI properties results in higher response than when using FD properties. When using FD shear modulus and damping (Type 1), high frequency components are significantly filtered out and thus results in lower peak ground acceleration and spectral acceleration at periods lower than 0.3 sec, relevant to stiff

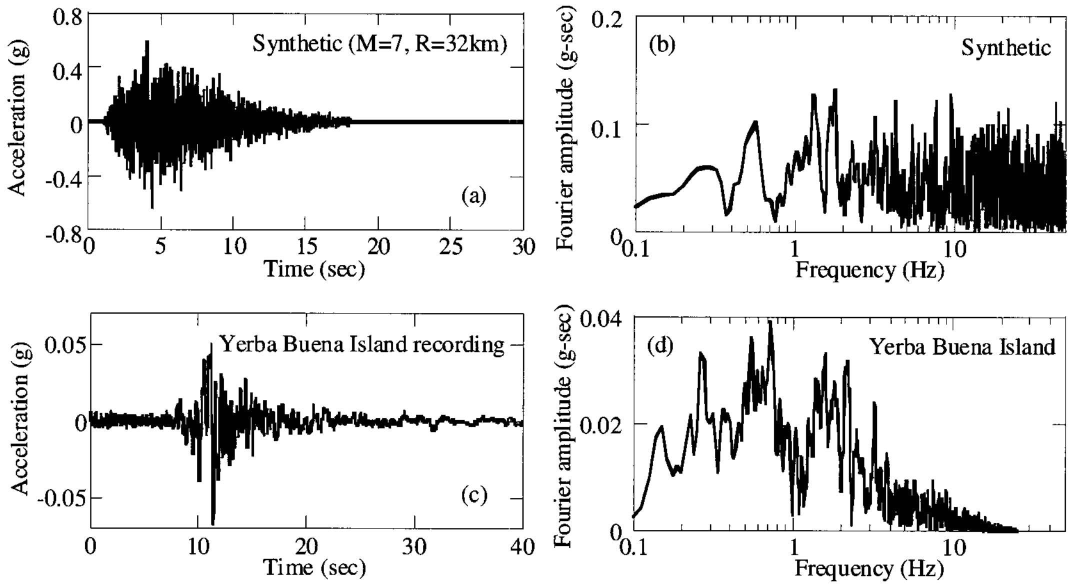

Fig. 12. Input motions used in site response analyses. The synthetic motion is generated by SMSIM (Boore, 2000) and the Yerba Buena Island motion is the recorded motion during the Loma Prieta earthquake, 1989.

structures such as low rise buildings.

When using FI shear modulus and Type 1 damping, the response is lower at short periods due to lower shear modulus at high frequencies assuming frequency independent behavior and higher at long periods due to higher modulus at low frequencies. However, the effect of the frequency dependent shear modulus is limited compared to the effect of frequency dependent damping ratio.

Fig. 14 shows the effect of various FD damping properties used in the analyses. Type 1 damping results in almost identical response to Type 2, even though Type 2 damping assumes no frequency dependent behavior at lower frequencies and thus results in lower damping. The main reason for the similarity is because of the low amplification at low frequency range, as can be observed from the transfer function (Fig. 14b). The natural period of the soil column is 1 Hz, and the Fourier spectrum ratio is 1 up to approximately 1 Hz and is almost identical for all types of FD damping. Type 1 damping results in higher attenuation compared to Type 3 damping due to higher damping at all frequencies.

Fig. 15 compares the results using FI properties and FD modulus and FD damping (Type 1). The input motion used is the recorded motion at Yerba Buena Island. The transfer function shows significant discrepancy, especially at periods lower than 0.3 sec (frequencies higher than 3.3 Hz). However, the computed surface response spectra are very similar, spectrum using FI properties resulting in slightly higher response. The main reason for the similarity is the frequency content of the input motion. Since the input motion has very low frequency content at frequencies lower or higher than 4 Hz, the difference in transfer function does not contribute much to the calculated motion.

Additional analyses are performed to characterize the effect of the shear wave velocity and thickness of the soil profile, as shown in Fig. 16. Fig. 16a shows results using 50 m thick profile with constant Vs of 500 m/sec. Compared to Fig. 15, higher Vs results in higher amplification of high frequency components and thus the effect of FD nature of soil damping is more pronounced. The effect

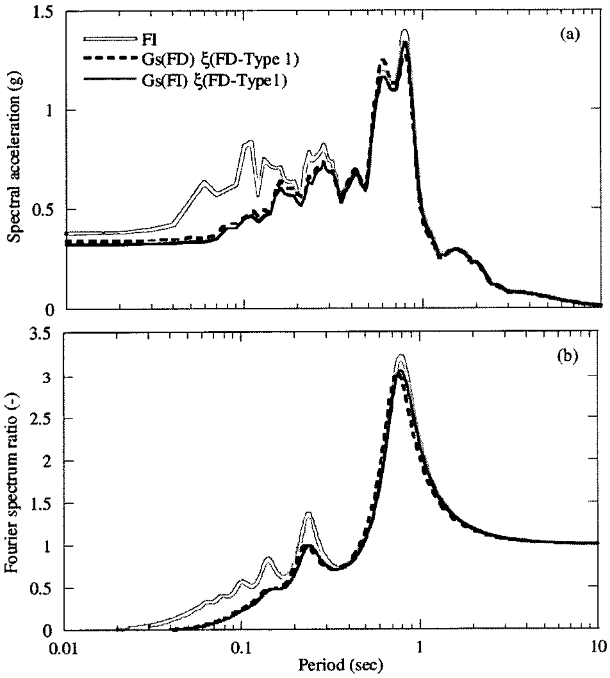

Fig. 13. Computed surface response spectra of 50 m profile, Vs = 200 m/sec, input motion: synthetic motion

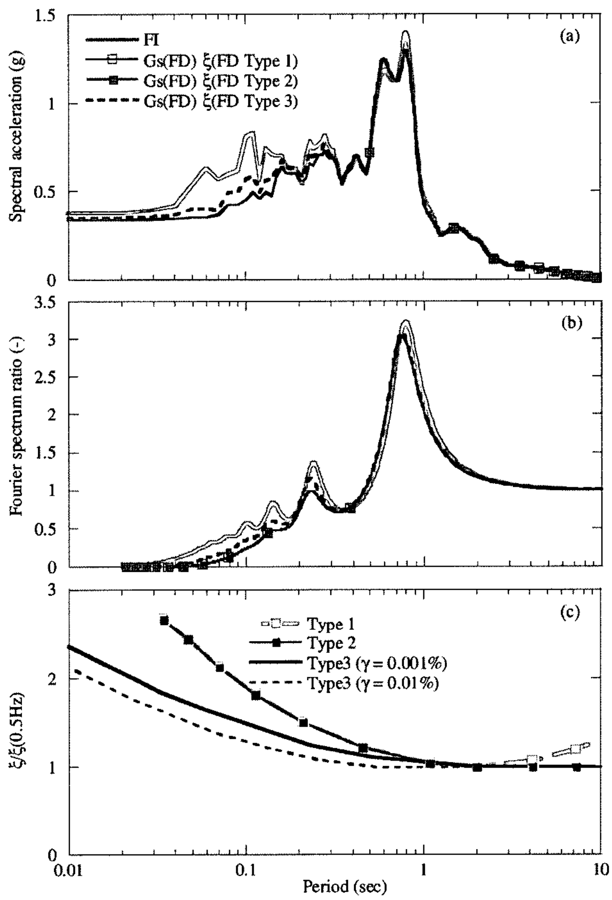

Fig. 14. Effect frequency dependent damping on computed surface response spectra of 50 m profile, Vs = 200 m/sec, input motion: synthetic motion

of the type of FD damping is important, Type 1 damping resulting in much lower response compared to Type 2 damping. The thickness of the soil profile, on the other hand, shifts the period of the surface spectrum to higher period. While the FD properties reduce response up to 0.1 - 0.2 sec for short soil column, the zone of influence is widened to up to 0.6 sec for the 300 m thick soil column, as shown in Fig. 16b.

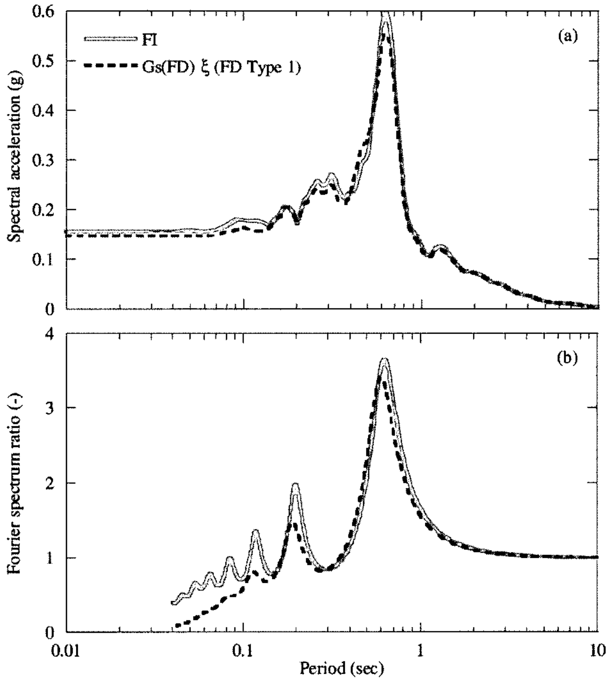

Fig. 15. Computed surface response spectra and transfer functions of 50 m profile, Vs = 200 m/sec, input motion: recorded motion at Yerba Buena Island

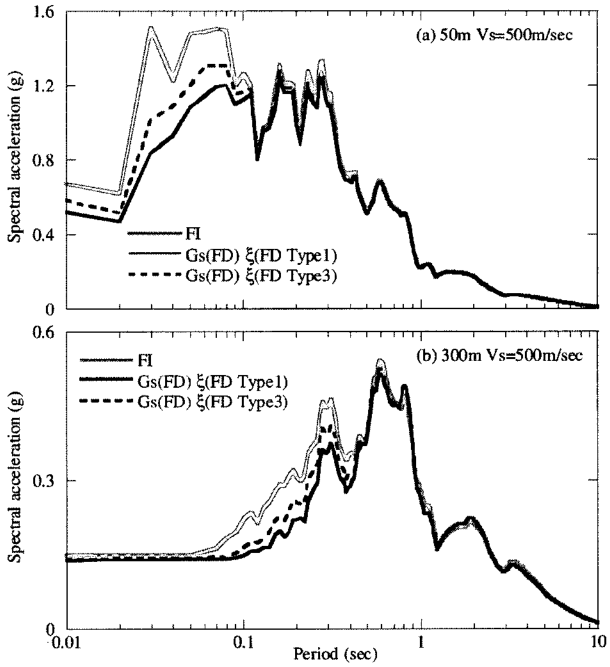

Fig. 16. Computed surface response spectra of (a) 50 m thick, Vs = 500 m/sec profile, and (b) 300 m thick, Vs = 500 m/sec profile, input motion: synthetic motion

Final example uses a 100 m thick profile with alternating layers of cohesionless and cohesive soils. This example is performed to illustrate the effect of the frequency dependent soil behavior in inhomogeneous soil column with varying layers of frequency dependent and independent soil behavior. The dynamic properties used in the analysis are also shown in Fig. 17. The sand layers are represented by average cohesionless curves developed by Seed et al. (1986). The cohesive layers are represented by soil curves used in previous analyses. Two analyses are performed, one assuming that only cohesive layers are FD, while the other analysis assumes that all layers are FD. The computed surface response spectra are shown in Fig. 18. As expected, the computed spectra assuming that all layers are FD result in higher response than when assuming that only clay layers are FD. The example clearly demonstrates that the loading frequency dependent nature of soil behavior cannot be ignored in profiles with clay layers and should be accounted for in performing site response analysis.

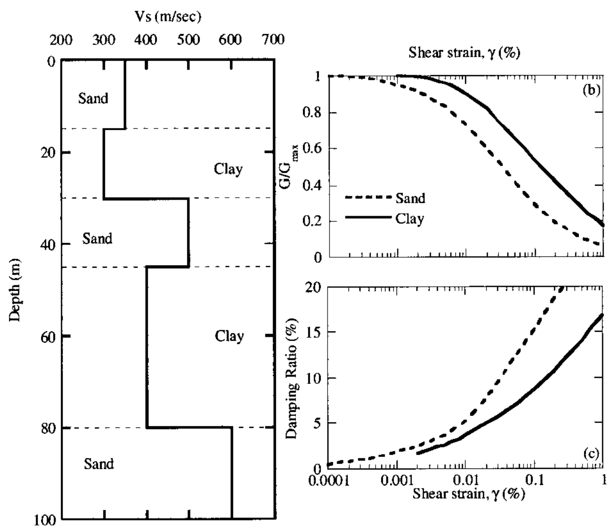

Fig. 17. Idealized profile of alternating layers of sands and clays and dynamic properties used in the analysis. The dynamic curves used are average curves for cohesionless soils developed by Seed et al. (1986) and PI=30 curves developed by Vucetic and Dobry (1991).

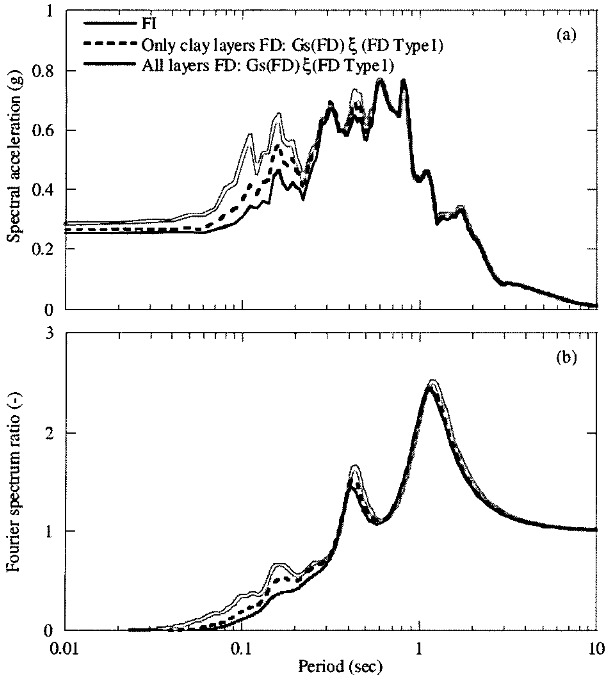

Fig. 18. Computed surface response spectra, variable Vs profile, input motion: synthetic motion. The dotted lines represent results assuming that clay layers are loading frequency dependent, while the sand layers are frequency independent. Solid black lines represent results assuming all layers (both sand and clay layers) are frequency dependent.

6. Conclusions

Laboratory tests show that the loading frequency greatly alters soil behavior, both shear modulus and damping ratio increasing with increase in loading frequency. A new equivalent linear analysis procedure that can account for the effect of loading frequency on soil behavior is developed. The procedure is capable of modeling both frequency dependent shear modulus and damping ratio. A series of modified equivalent linear analyses are performed to characterize the effect of the loading frequency dependent soil behavior on site response analysis. Results indicate that the loading frequency dependent nature of the shear modulus has limited influence on propagation of the seismic waves. However, the frequency dependent damping ratio has pronounced influence on propagated ground motion, filtering out high frequency components due to the higher energy dissipation at high loading frequencies and also resulting in lower peak ground acceleration. The effect is especially important when propagating ground motions rich in high frequency content.

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