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Estimation of Nonlinear Site Effects of Soil Profiles in Korea

국내 지반에서의 비선형 부지효과 예측

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요 지

시간영역에서 수행되는 비선형 지반응답해석에서 지반의 미소변형률 감쇠는 Rayleigh 감쇠공식을 이용하여 점성감쇠로서 모사된다. 실제 지반의 미소변형률 감쇠는 주파수의 영향을 받지 않는 반면 시간영역해석에서의 점성감쇠는 주파수의 영향을 크게 받으며 이의 영향정도는 Rayleigh 감쇠공식에 따라서 결정된다. 본 연구에서는 국내 지반에 대한 비선형 지반응답해석시 감쇠공식의 영향을 평가하고자 일련의 해석을 수행하였다. 해석결과 점성감쇠공식은 계산된 응답에 매우 큰 영향을 미치는 것으로 나타났다. 널리 사용되는 Simplified Rayleigh 공식은 심도 30m 이상의 지반에서 수치적으로 발생하는 인공감쇠로 인하여 고주파수에서의 에너지 소산을 과대예측하는 것으로 나타난 반면, Full Rayleigh 공식을 사용하며 적절하게 최적주파수를 선정한 경우, 인공감쇠는 크게 감소하는 것으로 나타났다. 나아가 해석결과를 등가선형해석과 비교한 결과 20m 미만의 얕은 심도 지반에서도 등가선형해석은 최대가속도를 과대예측할 수 있는 것으로 나타났다.

Abstract

In a nonlinear site response analysis which is performed in time domain, small strain damping is modeled as viscous damping through use of various forms of Rayleigh damping formulations. Small strain damping of soil is known to be independent of the loading frequency, but the viscous damping is greatly influenced by the loading frequency. The type of Rayleigh damping formulation has a pronounced influence on the dependence. This paper performs a series of nonlinear analyses to evaluate the degree of influence of the viscous damping formulation on Korean soil profiles. Analyses highlight the strong influence of the viscous damping formulation for soil profiles exceeding 30 m in thickness, commonly used in simplified Rayleigh damping formulation overestimating energy dissipation at high frequencies due to artificially introduced damping. When using the full Rayleigh damping formulation and carefully selecting the optimum modes, the artificial damping is greatly reduced. Results are further compared to equivalent linear analyses. The equivalent linear analyses can overestimate the peak ground acceleration even for shallow profiles less than 20 m in thickness.

Keywords : Equivalent linear, Nonlinear, Peak ground acceleration, Site response analysis, Viscous damping

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1. Introduction

One-dimensional (1-D) site response analysis is widely performed to estimate local site amplification effects during an earthquake (Hashash and Park, 2001; Idriss, 1990; Roesset, 1977), in which ground motion propagation is approximated as vertically propagating horizontal shear waves through horizontally layered soil deposit. Solution of wave propagation is performed in either frequency or time domain.

Equivalent linear analysis, performed in frequency domain, is dominantly used in practice due to its simplicity and ease of use (Schnabel et al., 1972). The equivalent linear method approximates nonlinear behavior by incorporating shear strain dependent shear modulus and damping curves. However, a constant linear shear modulus and damping at a representative level of strain are used throughout the analysis.

In a nonlinear analysis, the dynamic equation of motion is integrated in time domain and the nonlinear soil behavior is accurately modeled. However, non-linear site response analysis formulation contains the viscous damping term to model damping at small strains that does not always provide an accurate result. The influence of the viscous damping formulation has been known to be important for deep profiles thicker than 50 - 100 m (Hashash and Park, 2002). The influence of the formulation in shallower profiles has not yet been thoroughly studied. A series of nonlinear site response analyses are performed to investigate the influence of the viscous damping formulation at selected sites in Korea, ranging in thickness from 20 to 50 m. Results are further compared to equivalent linear analyses.

2. Nonlinear Site Response Analysis

In a nonlinear site response analysis, the response of a soil deposit is calculated by numerically integrating the wave propagation equation. Each individual layer i is represented by a corresponding mass, a spring, and a dashpot for viscous damping. Lumping half the mass of each of two consecutive layers at their common boundary forms the mass matrix. The stiffness matrix is built from the constitutive model and updated at each time step to simulate the nonlinear soil behavior.

In a nonlinear analysis, the hysteretic damping is modeled through the nonlinear soil model. Most nonlinear soil models display linear behavior at small strains, while the laboratory tests show that the soils damp the vibration even at very low strains. The small strain damping, which represent the damping ratio of the damping curve at the lowest strain level, is modeled by the viscous damping matrix [*C*]. Laboratory tests show that the small strain damping of cohesionless soil is independent of the loading frequency, while the cohesive soils are frequency dependent to a limited extent (Kim et al., 1991). However, for practical purposes, it is reasonable to assume that the small strain damping is frequency independent. In a time domain analysis, it is not possible to make the small strain damping independent of loading frequency.

The type of the damping formulation determines the degree of frequency dependence of the small strain damping. In the original damping formulation proposed by (Rayleigh and Lindsay, 1945), the [*C*] matrix is assumed to be proportional to the mass and stiffness matrix:

$$[C] = a_0[M] + a_1[K] \tag{1}$$

Scalar values of a_0 and a_1 can be computed using two significant natural modes m and n using the following equation:

$$\begin{bmatrix} \xi \\ \xi \end{bmatrix} = \frac{1}{4\pi}\begin{bmatrix} 1/f_m & f_m \\ 1/f_n & f_n \end{bmatrix}\begin{Bmatrix} a_0 \\ a_1 \end{Bmatrix} \tag{2}$$

where f_m and f_n are frequencies corresponding to selected modes m and n.

The damping matrix is assumed in most nonlinear seismic site response analysis codes to be only stiffness proportional (Borja et al., 2002; Matasovic and Vucetic, 1995), since the value of a_0[M] is small compared to a_1[K]. Small strain viscous damping effects are assumed proportional only to the stiffness of the soil layers. Such formulation will be termed simplified Rayleigh damping formulation (SF) and the original formulation will be termed full Rayleigh damping formulation (RF). Fig. 1 shows that SF results in a linear increase in damping with increase in frequency, and thus will introduce high numerical damping at frequencies higher than the natural mode of the soil column. The dependence of the damping on frequency is highly reduced using the RF.

The viscous damping formulation only matches the target frequency independent damping at one frequency for the SF and two frequencies for the RF. The formulation will either underestimate or overestimate the damping at other frequencies. The Rayleigh damping formulation can be extended so that more than 2 frequencies/modes can be specified, as shown in Fig. 1. However, incorporation of additional modes is accompanied by significant increase in computational cost, and the improvement in accuracy of the solution is limited. It is thus recommended that the full Rayleigh damping formulation be used in the analyses (Park and Hashash, 2004).

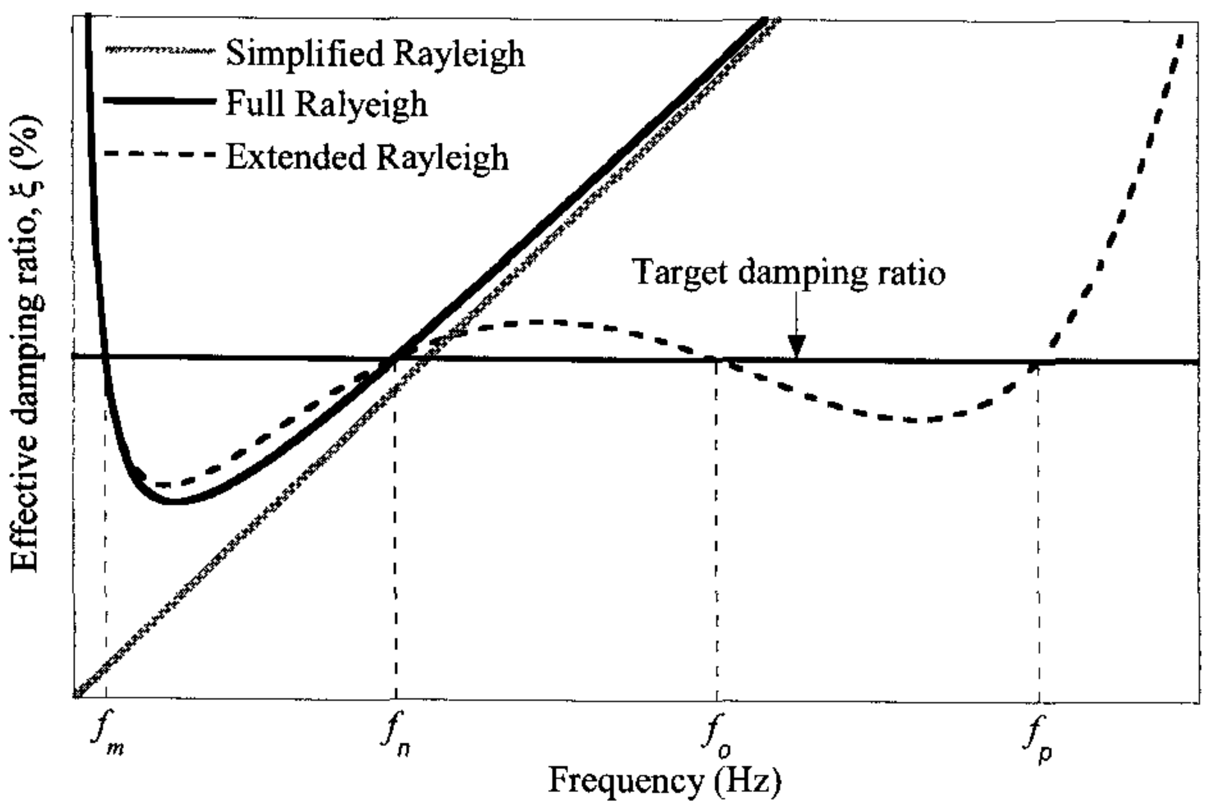


Fig. 1. Frequency dependence of simplified (SF), full Rayleigh damping formulation (RF), and extended Rayleigh damping formulation (Park and Hashash, 2004)

The effect of the frequency dependent nature of the viscous damping formulation is documented in Hashash and Park (2002) and Park and Hashash (2004) using a series of profiles up to 1000 m in thickness. It is concluded that the viscous damping formulation will introduce unacceptably high numerical damping for soil columns thicker than 50 - 100 m.

3. Site Description

The measured shear wave velocity profiles used in the analyses are shown in Fig. 2. The profiles are based on extensive site investigations performed in Korea (Kim et al., 2002; Sun et al., 2005; Yoon et al., 2006). Among 29 measured soil profiles selected in this study, 17, 15, and 7 profiles are classified as Site Class C, D, and E, respectively, according to the seismic design guideline (Ministry of Construction and Transportation, 1997). Most

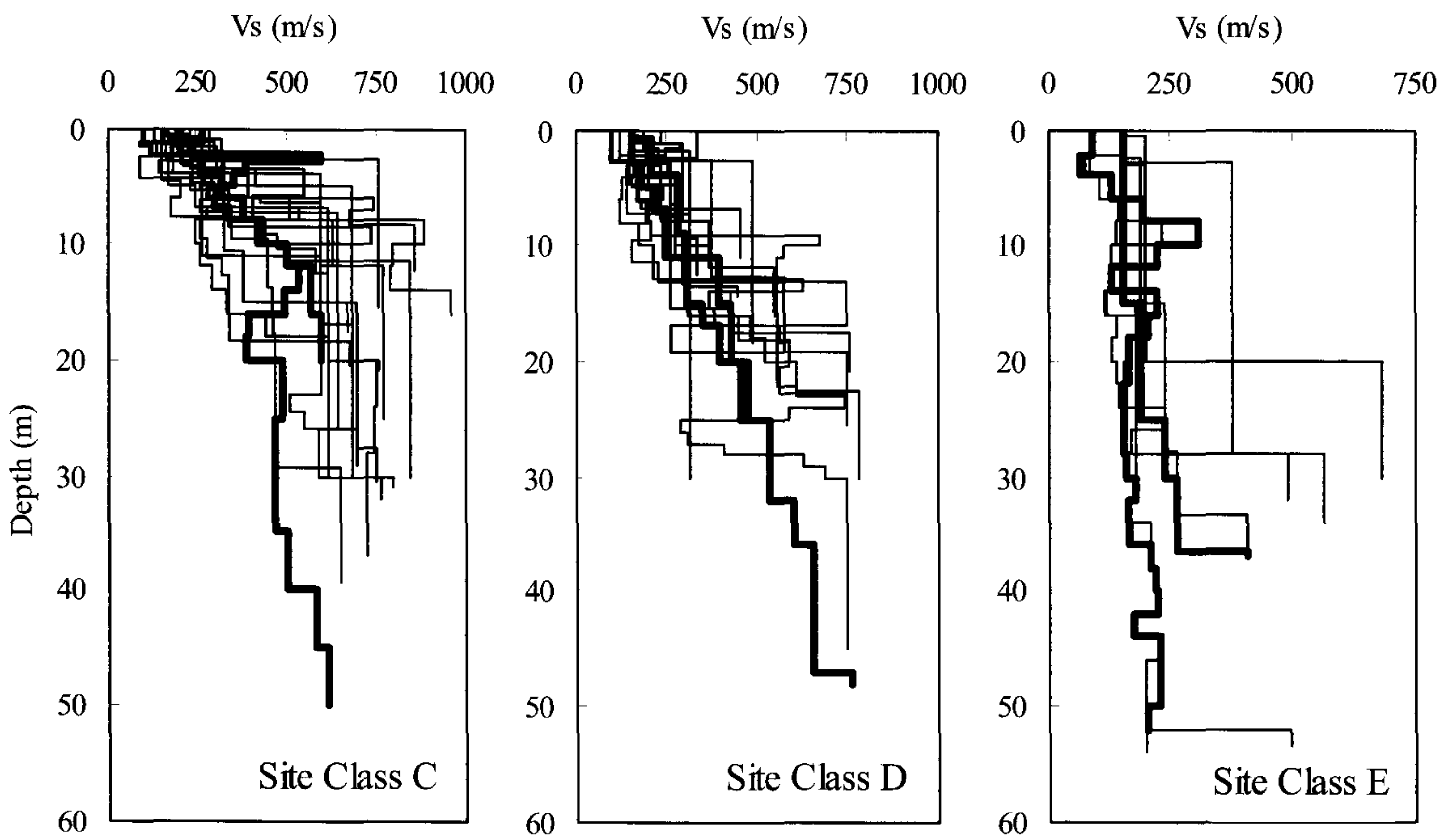


Fig. 2. Shear wave velocity profiles used in the site response analyses (Yoon, 2007)

inland profiles in Korea are classified as Site Class C or D. Site Class E profiles are located mostly in the coastal areas. Fig. 3 shows the site periods and thicknesses of all selected profiles. Site Class C profiles show the shortest site periods, ranging from 0.16 to 0.34 sec. Site Class D profiles range from 0.2 to 0.54 sec. Site Class E profiles show the longest site periods, ranging from 0.62 up to 1.1 sec.

The shear modulus reduction and damping curves selected for the analyses are shown in Fig. 4. The sedimentary soils and weathered soils used for the analyses are based on the resonant column tests of reconstituted samples of Gyeongju and undisturbed samples of Hongsong (Kim et al., 2002; Sun et al., 2002). The curves developed by Vucetic and Dobry (1991) for PI=30 soils are used for clays.

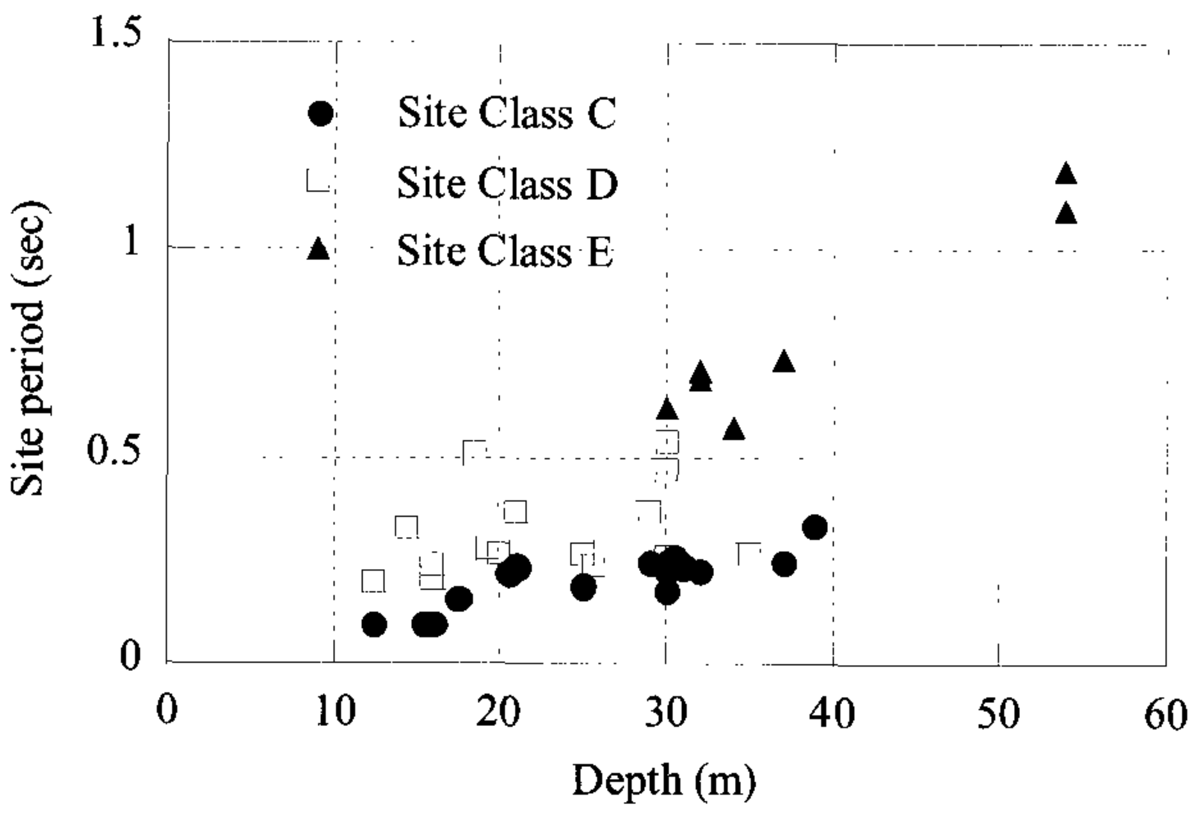


Fig. 3. Site periods and depths of the soil profiles

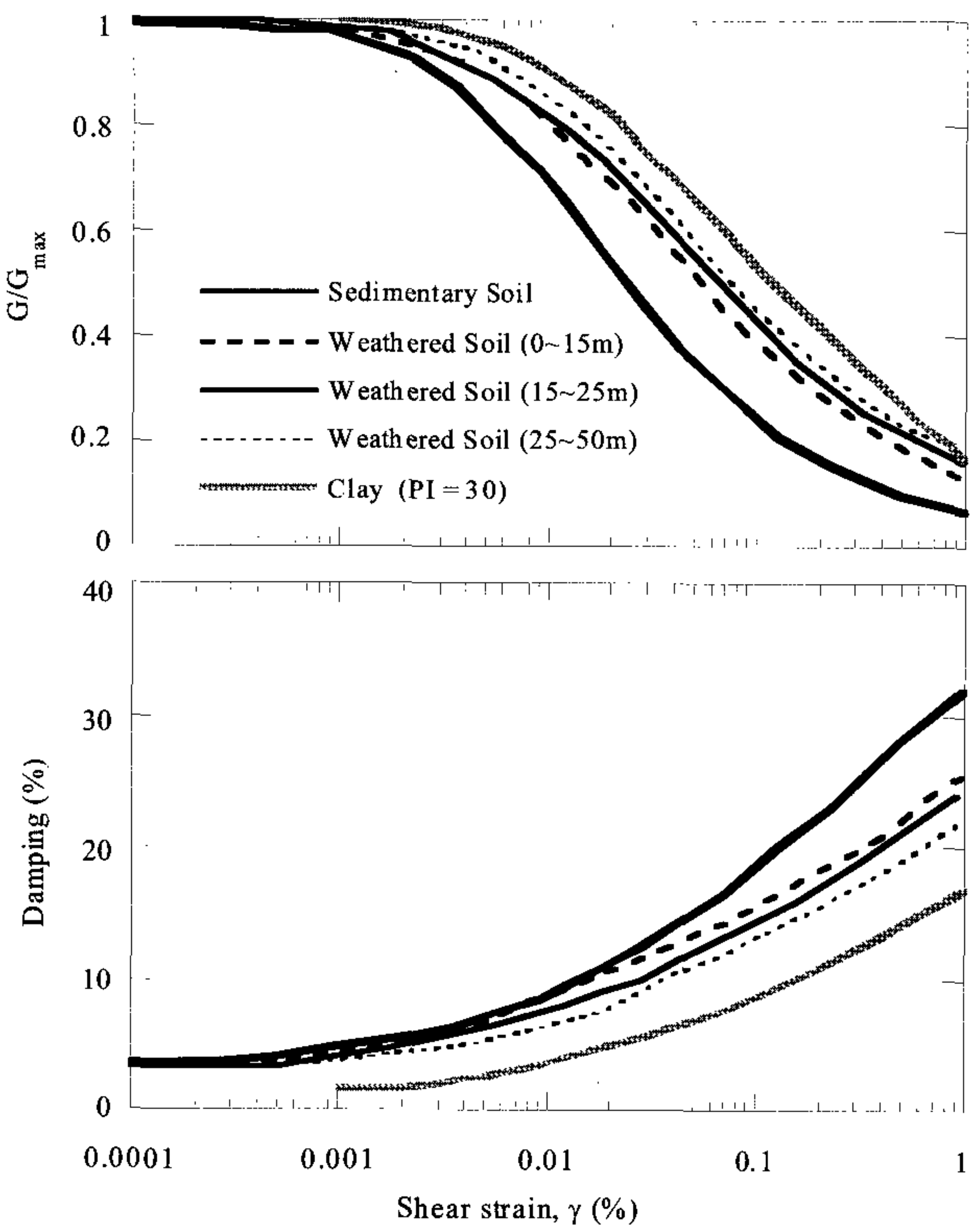


Fig. 4. Dynamic curves obtained used in the analyses

4. One Dmensional Nonlinear Site Response Analysis

A series of nonlinear site response analyses are performed at the selected sites to characterize the effect of the viscous damping formulation. Korean seismic code only defines the seismic hazard in terms of peak ground acceleration (PGA). Korea is divided into two seismic zones based on probabilistic seismic hazard analysis, termed zone I and II. The PGA of the seismic zone I for earthquakes with return periods of 1000 years (equivalent to 10% probability of exceedance in 10 years) and 2400 years (equivalent to 10% probability of exceedance in 250 years) are 0.154 g and 0.22 g, respectively. The PGA of seismic zone II for return periods of 1000 years and 2400 years are 0.1 g and 0.14 g, respectively. In this study, all sites selected are assumed to be in seismic zone I.

Three motions are used in the analyses, as shown in Fig. 5. The first motion is the recorded motion at Yerba Buena Island during Loma Prieta earthquake (U.S., M=7.1, PGA=0.067 g). The second motion is the recorded motion at Ofunato during Miyugi-Oki earthquake (Japan, M=7.4, PGA=0.23 g). The third motion is a synthetic motion developed using SIMQKE (Gasparini and Vanmarcke, 1976), which is widely used to develop response spectrum compatible ground motions in Korea. Each of the selected motions has been scaled to match the PGA of seismic zone I, with return periods of 1000 and 2400 years. Note that the acceleration time histories and Fourier spectra of the input motions shown in Fig. 5 are scaled to a PGA of 0.154 g. Even though the motions are representative of the ground motions at rock outcrop, the frequency characteristics show distinct variation. The motion recorded at Yerba Buena Island is rich in low frequency and relatively low in high frequency. The recorded motion at Ofunato (dominant frequency = 3 Hz) is rich in high

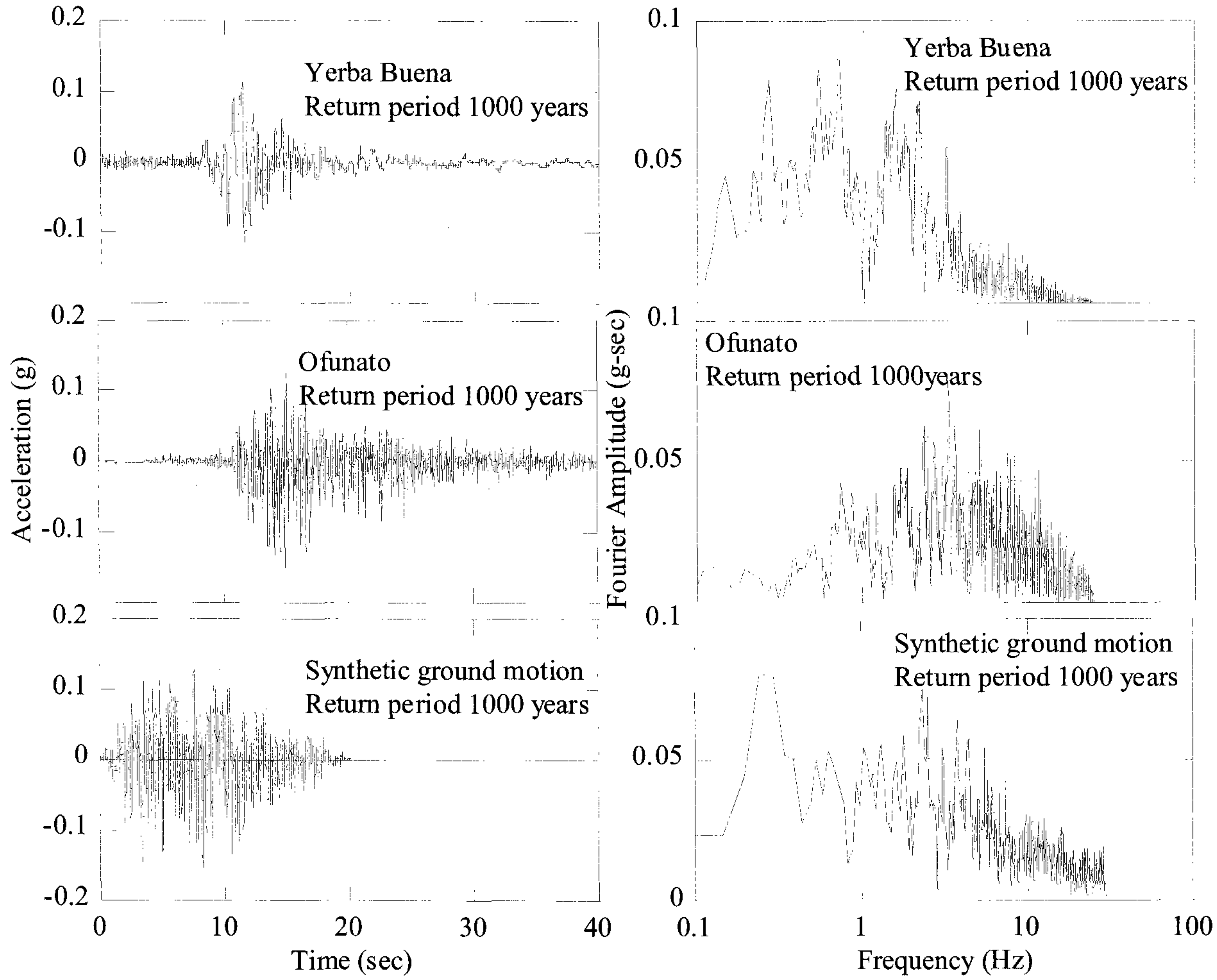


Fig. 5. Time histories and Fourier spectra of the input motions

frequency and very low at frequencies between 0.1 and 1 Hz. The energy of the synthetic motion is evenly distributed along the full frequency spectrum. Fig. 6 shows the 5% damped acceleration response spectra of the input motions. When comparing the response spectra of the input motions with the design spectrum, the Ofunato and synthetic motions match very well with design spectrum, while the Yerba Buena motion is lower at short periods and higher at long periods.

Six profiles are selected to be used in the analyses, two for each Site Class. The selected profiles are shown as thick lines in Fig. 2. Nonlinear analyses are performed using the one dimensional site response analysis code newly developed code GEOSHAKE. GEOSHAKE is built upon DEEPSOIL (Hashash and Park, 2001), with various additional features including rate dependent soil modeling and frequency dependent equivalent linear algorithm. However, such features are not used in the analysis. The

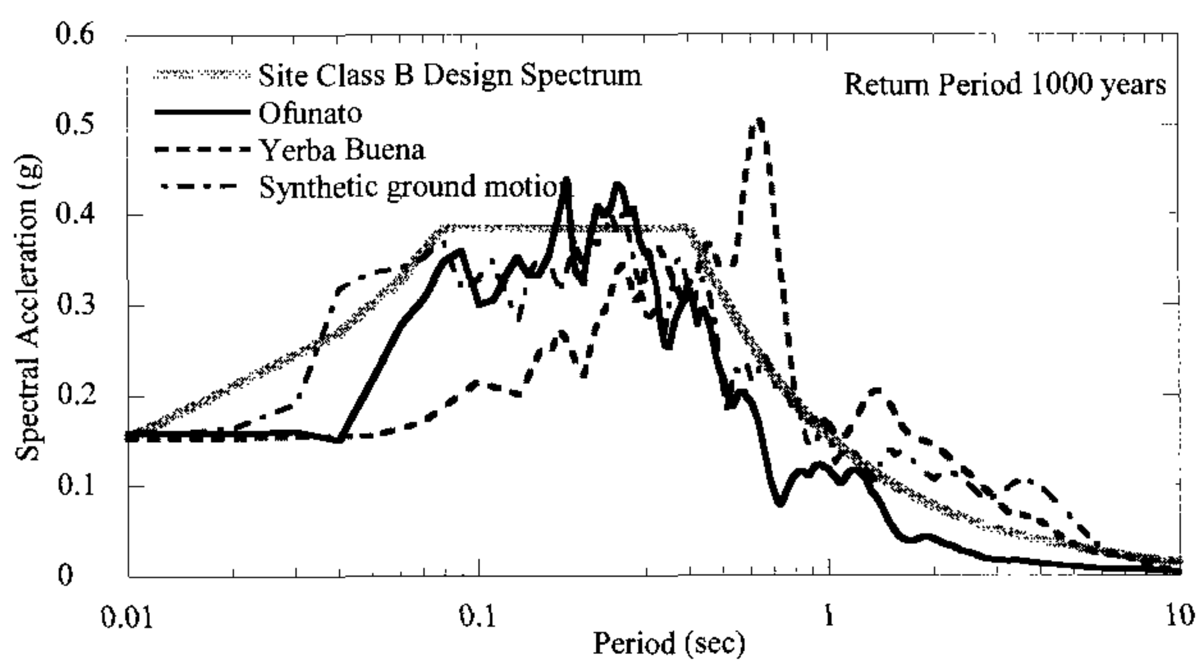
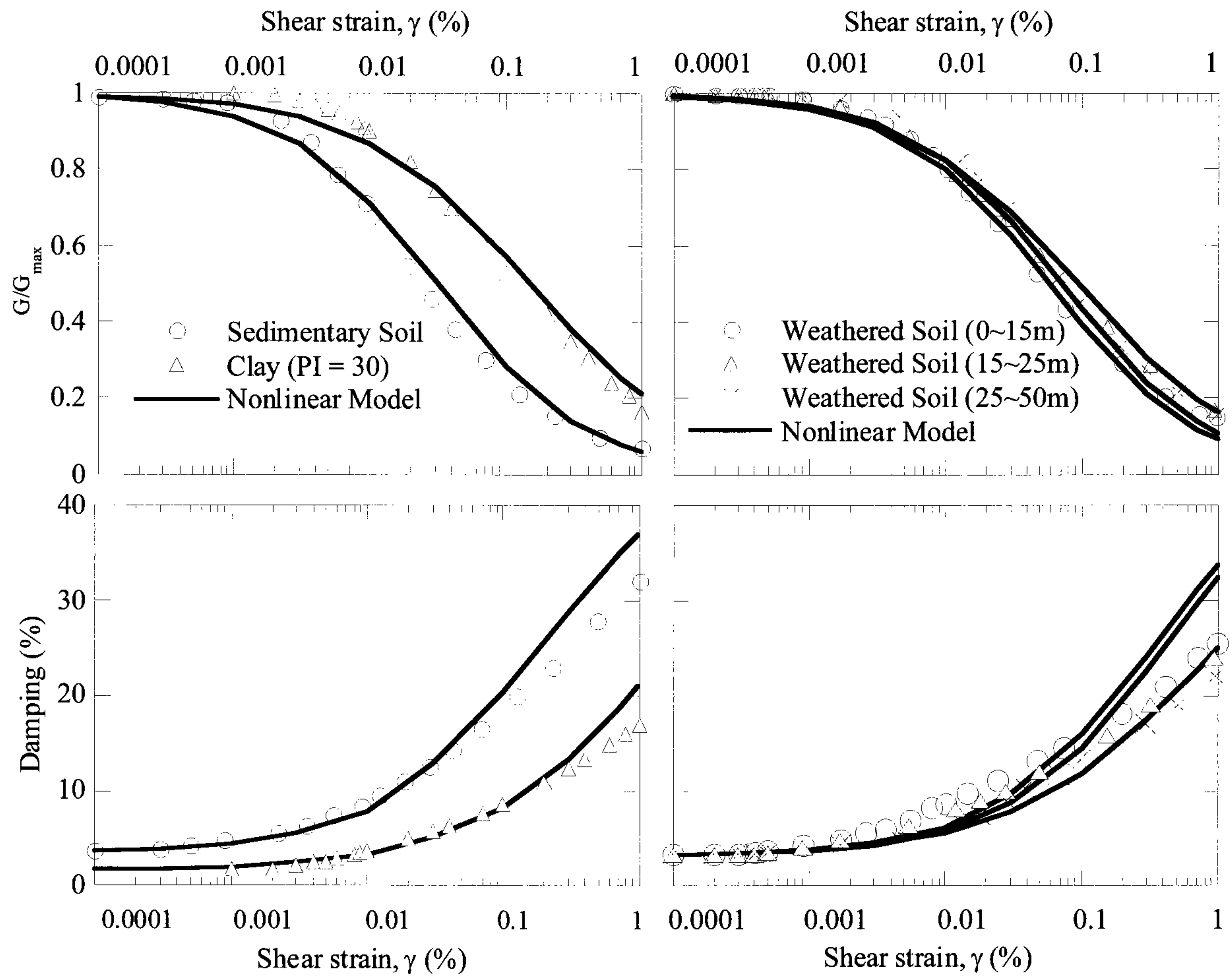


Fig. 6. Response spectra of the scaled input motions and the design response spectrum

constitutive model used in GEOSHAKE is the modified hyperbolic model (Matasovic, 1993), which is defined as follows

$$\tau = \frac{G_{mo}\gamma}{1 + \beta\left(\frac{\gamma}{\gamma_r}\right)^s} \tag{3}$$



Soil Type	β	s	γ_r(%)
Sedimentary Soil	1	0.8	0.03
Clay	0.15	0.7	0.01
Weathered Soil (0-15m)	0.6	0.8	0.03
Weathered Soil (15-25m)	0.6	0.8	0.04
Weathered Soil (25-50m)	0.6	0.8	0.05

Fig. 7. Comparison of the measured data (shown as discrete points) and the shear modulus and damping curves derived from the nonlinear constitutive model (shown as solid lines)

where τ = shear stress, γ = shear strain, G_{mo} = maximum shear modulus, γ_r = reference strain ($\gamma_r = G_{m0}/\tau_{m0}$, τ_{m0} = shear strength), β and s = curve fitting parameters that adjust the shape of the backbone curve.

Fig. 7 compares the reference dynamic soils curves (shear modulus reduction and damping curves) with the curves derived from the nonlinear soil model. The shear modulus reduction curves from the nonlinear model match very well with the measured curves. The damping curves match reasonably well the measured curves except for the weathered soil, where the nonlinear model overestimates damping at strains higher than 0.1%. The optimum modes for the RF are selected based on the guidelines proposed by Park and Hashash (2004). The predominant site frequency is selected for the SF.

Fig. 8 compares the results of nonlinear analyses using the SF and RF. For soil columns less than 30 m in thickness, the responses using the SF and RF are within tolerable range. However, for soil columns exceeding 30 m in thickness, the influence of the viscous damping formulation is pronounced. The influence of the viscous damping increases with increase in the thickness of the soil column. If the thickness of the soil column increases, the predominant site frequency decreases (f_m in Fig. 1

decreases). It will result in overestimating the damping for all frequencies higher than f_m. Therefore, SF results in significantly lower response than the RF. The results demonstrate that the use of the SF should not be permitted for soil profiles thicker than 30 m.

Table 1 lists the selected optimum modes for the nonlinear site response analyses. It is evident that the modes are dependent on the frequency content of the ground motion and the site period of the soil profile. The selected modes for Ofunato motion is 1^{st} and 3^{rd}, 1^{st} and 5^{th}, and 1^{st} and 8^{th}. The selected higher mode increases with increase in the thickness and site period of the soil column. The selected modes for the Yerba Buena motion are different from the modes selected for the Ofunato motion, which are 1^{st} and 3^{rd} and 1^{st} and 4^{th}. The selected higher mode is lower than when using the Ofunato motion because the Yerba Buena motion has low energy content at high frequencies. The selected modes are very similar for the synthetic motion, ranging from 1^{st} and 2^{nd} to 1^{st} and 4^{th}. Although there is a tendency for the selected

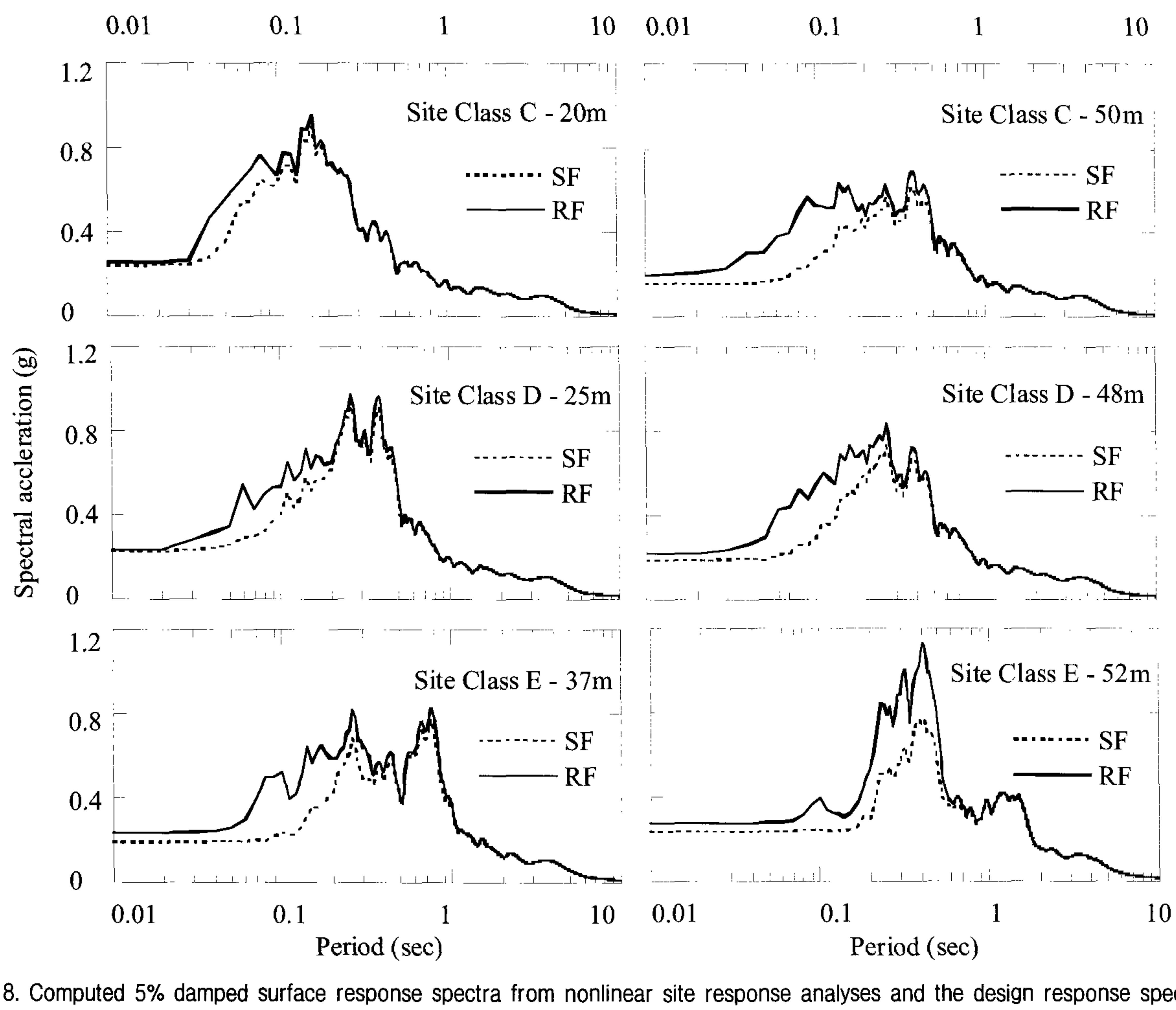


Fig. 8. Computed 5% damped surface response spectra from nonlinear site response analyses and the design response spectrum

Table 1. Selected modes for the full Rayleigh damping formulation

Site Class	Thickness (m)	Natural Period (sec)	Selected modes		
			Ofunato motion	Yerba Buena motion	Synthetic motion
C	20	0.18	1^{st} & 3^{rd}	1^{st} & 3^{rd}	1^{st} & 2^{nd}
	50	0.42	1^{st} & 5^{th}	1^{st} & 4^{th}	1^{st} & 4^{th}
D	25	0.32	1^{st} & 5^{th}	1^{st} & 3^{rd}	1^{st} & 3^{rd}
	48	0.40	1^{st} & 8^{th}	1^{st} & 4^{th}	1^{st} & 4^{th}
E	37	0.74	1^{st} & 8^{th}	1^{st} & 4^{th}	1^{st} & 3^{rd}
	52	1.13	1^{st} & 8^{th}	1^{st} & 4^{th}	1^{st} & 3^{rd}

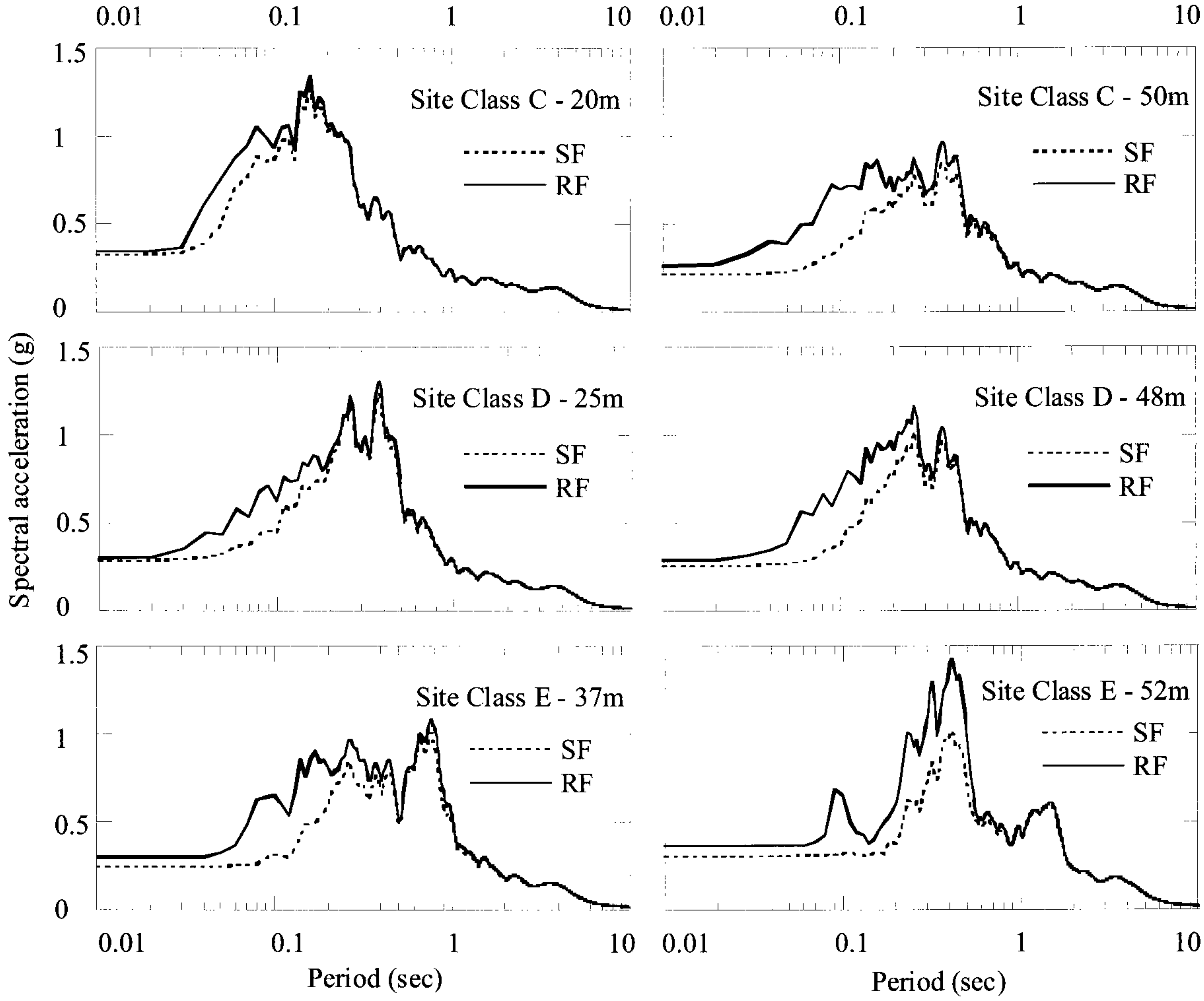


Fig. 9. Computed 5% damped surface response spectra from nonlinear site response analyses and the design response spectrum

higher mode to increase with increase in the thickness of the soil column, there is no clear rule in selecting the modes and thus have to be selected by trial and error.

Fig. 9 compares the computed response spectra using the synthetic motion scaled to PGA = 0.22 g. The selected modes are identical to those selected for ground motion scaled to PGA = 0.15 g. The discrepancy between SF and RF is very similar to Fig. 8. For the profiles and ground motions used in this study, the modes are independent of the scaling of the ground motions.

5. Comparison of Nonlinear and Equivalent Linear Analyses

The use of equivalent linear analysis in Korea has been dominant, since the equivalent linear analysis is known to be reliable when performing analyses at shallow soil profiles and propagating weakly to moderate ground motions (Kramer, 1996). A series of nonlinear and equivalent linear analyses are performed using all soil profiles shown in Fig. 2 to determine the degree of discrepancy between the analysis methods. The equivalent linear analyses are also performed using GEOSHAKE.

In performing an equivalent linear analysis, the dynamic soil behavior is modeled using shear modulus reduction and damping curves, whereas a constitutive model is used in a nonlinear analysis. Current nonlinear constitutive models cannot exactly simulate the measured soil behavior. In addition, the curves used in practice are most often representative curves developing often an array of measurements. In such case, it is impossible to simulate the curves by a constitutive model. The differences between the representative curves and the curves derived from the nonlinear analysis have been shown in Fig. 7. Such approximation is acceptable for most purposes. However, since this comparison is intended to characterize the difference originating from the analysis procedure only, different characterization of the dynamic soil behavior is a source of discrepancy between two analysis methods and should not be allowed. Therefore, the measured shear modulus reduction and damping curves, Fig. 4, are not used in the equivalent linear analysis. Instead, the shear modulus and damping curves derived from the nonlinear constitutive model, Fig. 7, are used in the equivalent linear analysis.

Fig. 10 compares the computed 5% damped surface spectra between equivalent linear and nonlinear analyses for the 48 m thick Site Class D profile. When using the Ofunato motion, the calculated response spectra show distinct discrepancy, the equivalent linear resulting in higher estimation of the PGA and spectral acceleration between 0.2 to 0.4 sec. The calculated response from the equivalent linear analysis is also higher when using the synthetic motion, but the difference is more subtle. For both cases, the PGA calculated by the equivalent linear analysis is higher than the nonlinear analysis results. The reason for the overestimation of the PGA is due to the intrinsic limitation of the equivalent linear procedure. It is a common practice in performing an equivalent linear analysis to use the secant shear modulus and damping at 65% of the maximum shear strain. In such case, the simulated soil behavior becomes stiffer at maximum shear strain (Yoshida and Iai, 1998). The PGA is most likely be the highest at maximum shear strain. Since the soil behavior at maximum shear strain is stiffer, the PGA becomes larger than when modeling the true nonlinear behavior.

Fig. 11 compares the computed PGA ratios, which is defined as the ratio of the PGA from the equivalent linear to that from the nonlinear analyses, as functions of the site period. The range of calculated PGA ratio is from unity up to 1.5. Very few analyses resulted in ratios below unity, but even in such cases, the ratios are very close

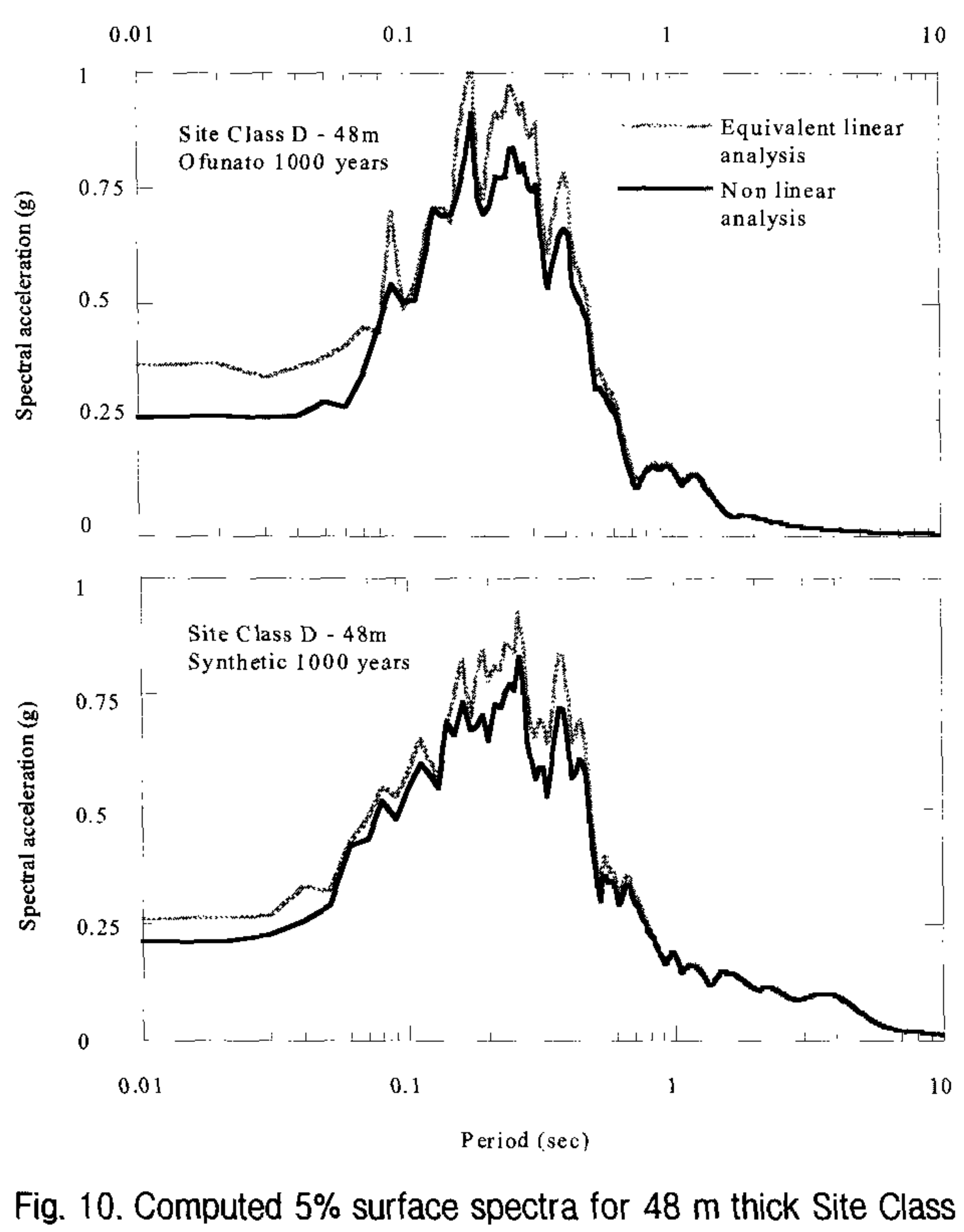


Fig. 10. Computed 5% surface spectra for 48 m thick Site Class D profile using Ofunato and synthetic motions

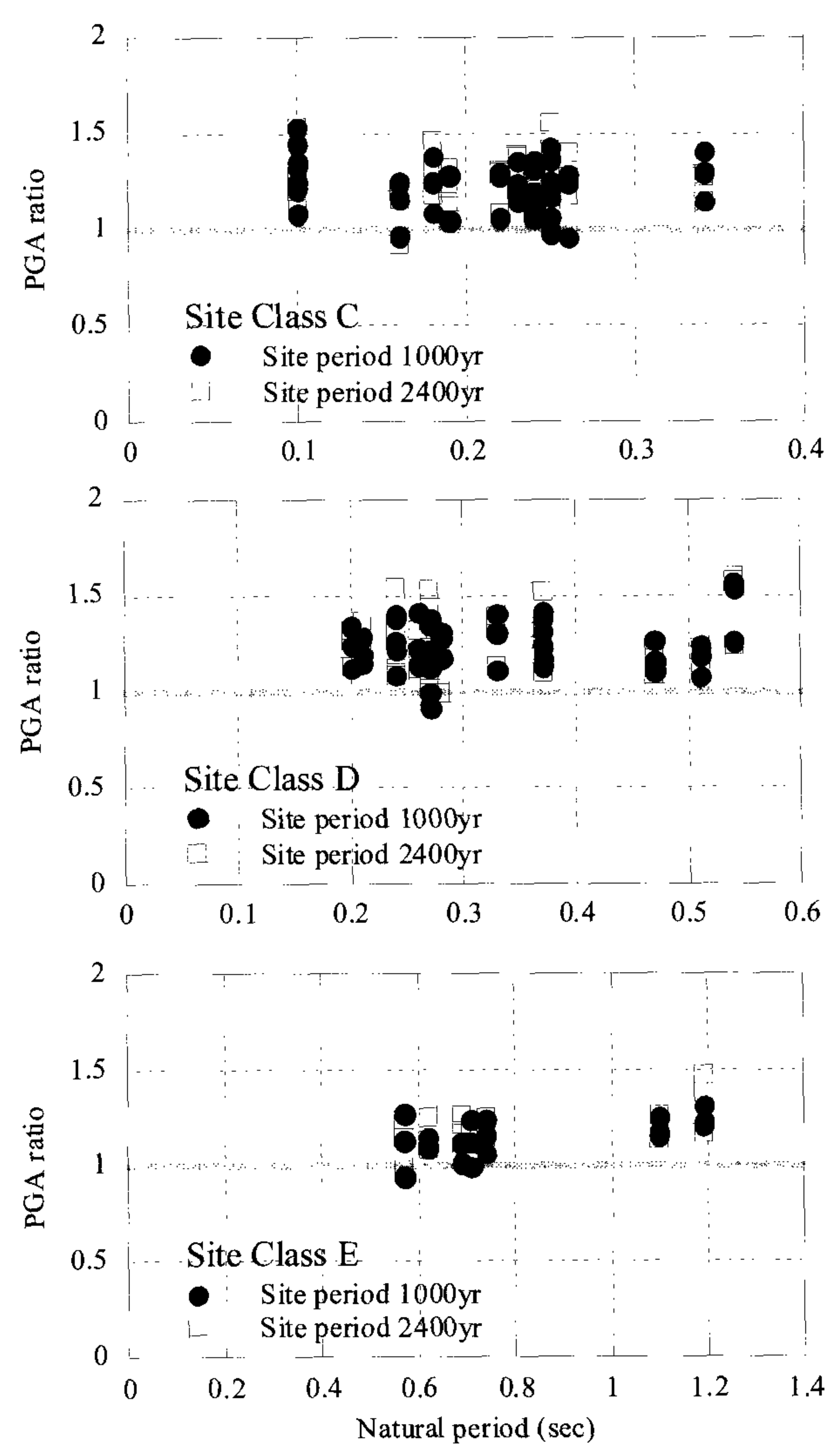


Fig. 11. Computed ratios of PGA from equivalent linear to those from nonlinear analyses

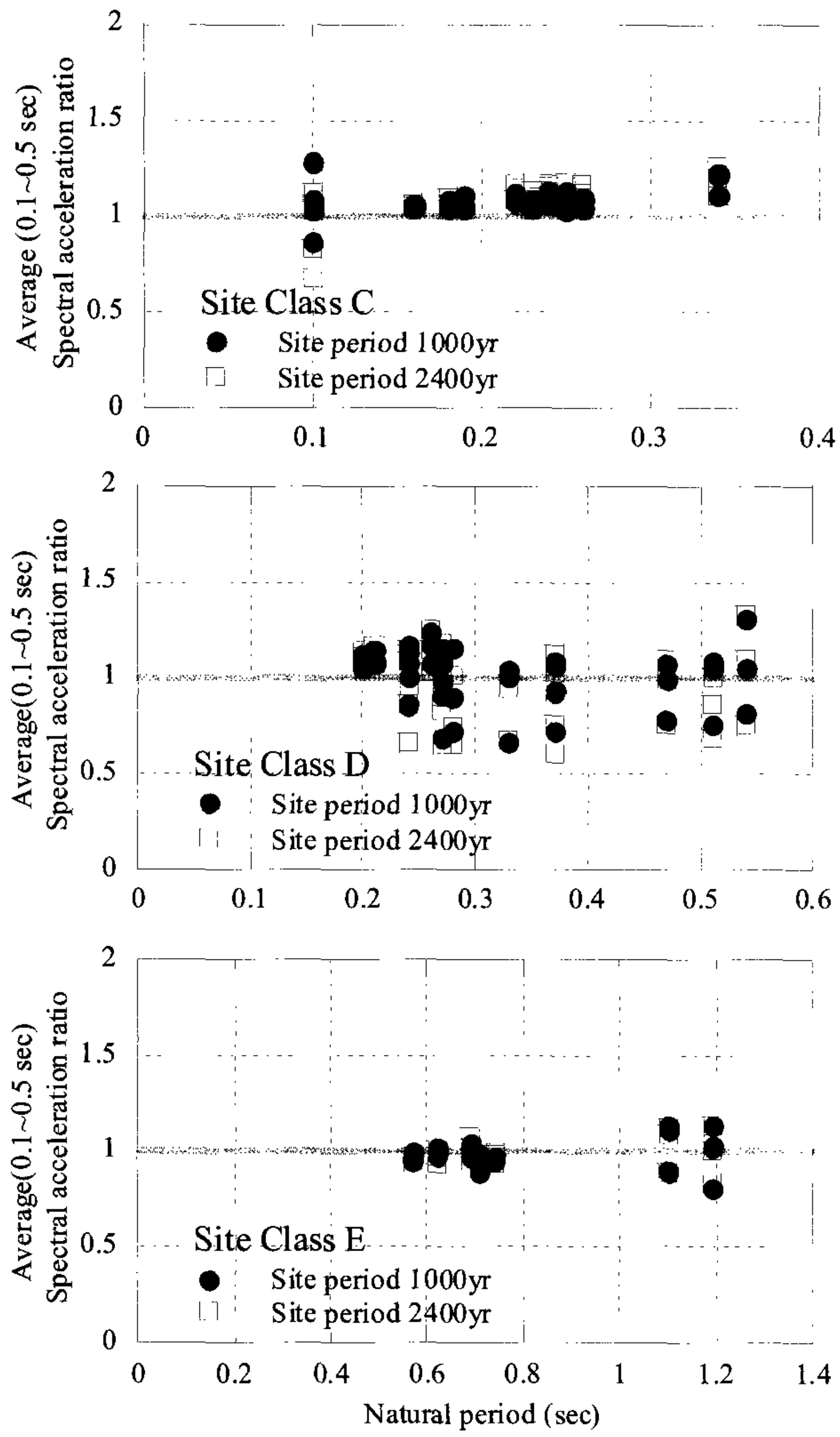


Fig. 12. Computed ratios of average spectral accelerations (0.1 – 0.5 sec) from equivalent linear to those from nonlinear analyses

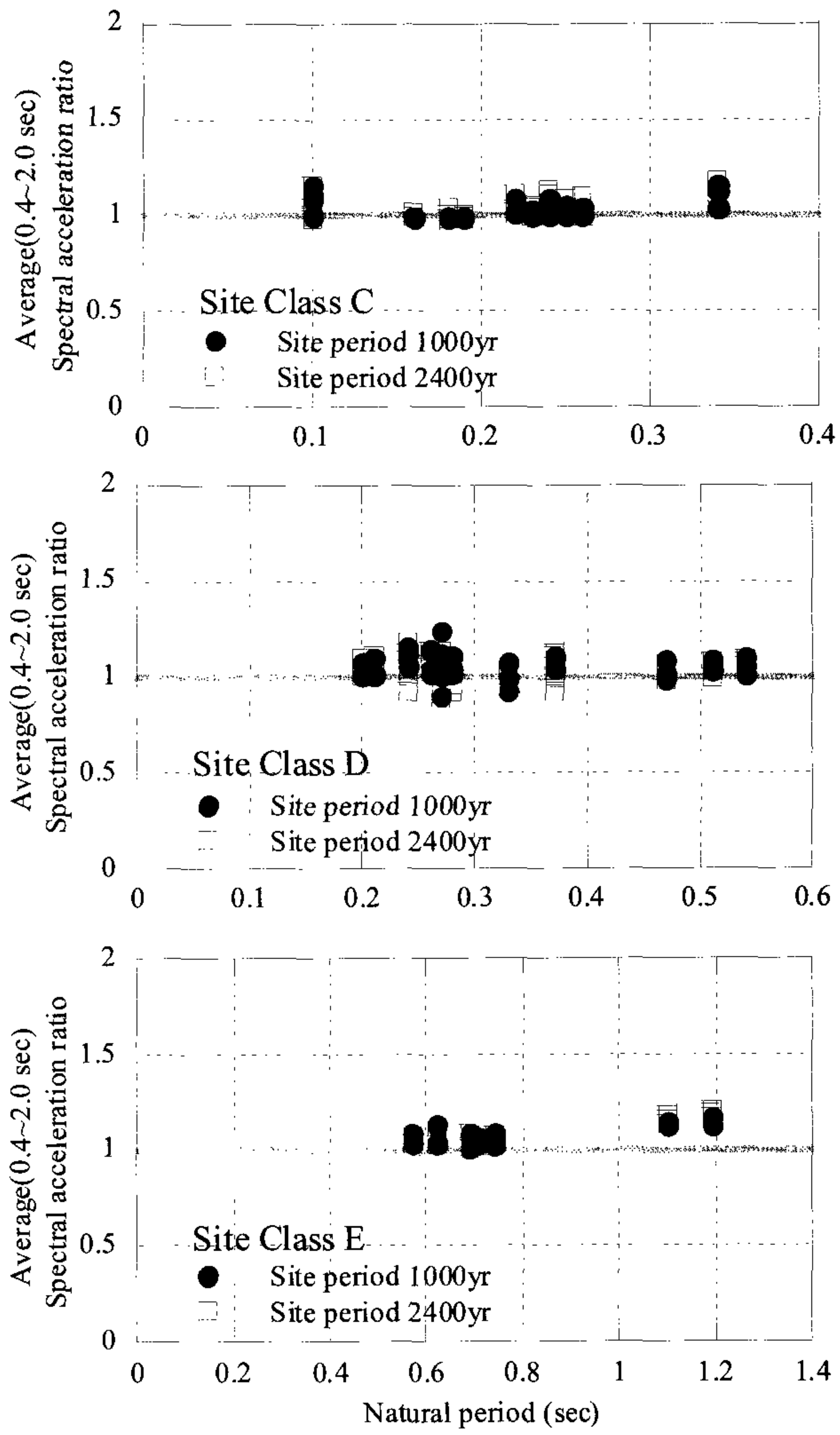


Fig. 13. Computed ratios of average spectral accelerations (0.4 – 2.0 sec) from equivalent linear to those from nonlinear analyses

to unity. It can thus be concluded that the equivalent linear analysis overestimates the PGA, while the degree of overestimation is variable. A clear dependence of the ratio on the input ground motion characteristics or the site period is not observed, resulting in significant scatter in the ratios. The ratio is, however, dependent on the amplitude of the ground motion. The PGA ratios using input motions scaled to a PGA of 0.22 g results are higher than those using motions scaled to 0.154 g. It is evident that the accuracy of the equivalent linear analysis decays for stronger ground motions. Fig. 12 and Fig. 13 compare the average spectral acceleration ratios between 0.1 to 0.5 sec and 0.4 to 2.0 sec. The average spectral acceleration between the 0.1 to 0.5 and 0.4 to 2.0 has been used to develop short-period and mid-period amplification factors (termed Fa and Fv, as defined in NEHRP Provisions). In contrast to the PGA, the equivalent linear analyses do not always display higher estimates compared to the nonlinear analyses. The comparisons demonstrate that the type of analysis has a more pronounced influence on the PGA than the average spectral accelerations.

6. Conclusions

A series of nonlinear site response analyses are performed at various measured soil profiles in Korea. Three input motions scaled to peak ground acceleration representative of seismic hazard with return periods of 1000 years and 2400 years are used. Analyses results demonstrate that the viscous damping formulation has pronounced influence on the propagated ground motion. The simplified Rayleigh damping filters out important frequency components even for soil profiles higher than 30 m in thickness. The effect becomes more significant with increases in thickness and decrease in stiffness of the soil profile. When using the full Rayleigh damping formulation and carefully selecting the optimum modes, the artificial damping introduced is greatly reduced. Results confirm the importance of controlling the viscous damping in a nonlinear analysis and that the use of the full Rayleigh damping and selecting optimum modes is not an option, but a prerequisite for obtaining reliable results for profiles higher than 30 m in thickness.

Results are further compared to equivalent linear analyses. Comparisons show that the computed PGA is highly dependent on the analysis type, the equivalent linear analyses consistently overestimating the response even for stiff and shallow soil columns.

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