STOCHASTICALLY BASED PREDICTION OF SITE-SPECIFIC SPECTRAL RESPONSE FOR DESIGN

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SUMMARY

The influence of local site conditions on the spectral response, as well as its interaction with the other predictor variables is a subject of ongoing research, where the need for incorporating the intensity of input motion in the prediction of site-specific response estimates is particularly emphasized. This study, conceived on a random vibration methodology basis, is a contribution to this research trend. Its global objective is the formulation of a stochastic model for the evaluation of site-specific response estimates in an approximate but acceptable form for practical design purposes, besides the particular objective of examining the simulation capabilities of the proposed model through the comparison with already in use design approaches. Results presented in this paper indicate that the proposed stochastic model can be used for estimating input motion intensity dependent site-specific spectral amplification factors for design purposes and improving site-dependent building-code provisions.

INTRODUCTION

It is well known that earthquake motion is dependent on the soil deposit characteristics and consequently much research has already been done on this topic in order to predict the earthquake hazard at a site and possibly its specific response spectra. Furthermore, in most of seismic design codes the effect of local surface soil conditions is qualitatively considered through the soil condition related dynamic response coefficient, but still on this point more research is required for a more reliable quantitative estimation of ground motion characteristics and structural response at a given site. The influence of local site geology on the free field ground motion characteristics and spectral response, as well as their interaction with the other predictor variables is a subject of ongoing research [Sugimura et al., 1991; Sugito et al., 1991; Borcherdt, 1994], where the need for incorporating the intensity of excitation in the prediction of site-dependent response spectra is particularly emphasized.

Since for most of the seismic regions of the world and the different site soil conditions in each region there are no representative ground motions at the various intensity levels required for proper earthquake resistant design, a predictive model is needed. Such a model should be conceived in a way that its predictions be acceptable for practical design purposes. This study, based on a theoretical earthquake source model and random vibration theory concepts, is a contribution to this research trend. It focuses first on the formulation of a stochastic model for the evaluation of site-specific response estimates and then follows with the examination of simulation capabilities of the proposed model through the comparison with already in use design approaches.

EARTHQUAKE EXCITATION MODELING

The following modified Markov spectrum [Aoyama and Matsushima, 1993], representing the double-sided incident wave acceleration power spectral-density function SA(ω) and given by

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$$S_A(\omega) = S_0 \cdot \{\frac{(\omega/\omega_c)^2}{1+(\omega/\omega_c)^2}\}^2 \cdot \frac{1}{1+(\omega/\omega_m)^2} \tag{1}$$

is assumed as input motion at the engineering bedrock level. The engineering bedrock is defined here as the stratum having shear wave velocity larger than 400m/s. S_0 represents the level of power spectral-density, while ω_c and ω_m represent two corner frequencies. The latter is related to the so called ω_{max}, which appears in the observed strong ground motions. The middle term inside the parenthesis represents the source spectrum and corresponds to the widely used ω^{-2} model. It has the characteristics of a high-pass filter. The last term represents the Markov spectrum and has the characteristics of a low-pass filter.

The squared mean value of incident acceleration A can be obtained by the following integral:

$$\sigma_A{}^2 = \int_{-\infty}^{\infty} S_A(\omega)d\omega = 2\pi S_0 \omega_m \cdot \{\frac{2\xi^4 - 3\xi^3 + \xi}{4(1-\xi^2)^2}\} \quad ; \quad \xi = \omega_m / \omega_c \tag{2}$$

Parameters which appear in Eq. (1) have definite physical meaning and are affected by several factors, however considering the purpose of this study, the values of two corner frequencies, ω_c and ω_m, were estimated by making use of nonlinear least squares technique in fitting the smoothed average acceleration power spectrum of widely used strong ground motions recorded on the free field to a model [Myslimaj, 1998] obtained from the multiplication of the modified Markov spectrum by the well-known Tajimi spectrum. The values for the best-fit model parameters ω_c and ω_m were found to be $0.886s^{-1}$ and $47.04s^{-1}$, respectively.

Based on the equation (2), a relation between the level of power spectral-density S_0 and acceleration standard deviation at the engineering bedrock σ_A can be derived, having the latter as a variable parameter. If this parameter is to be correlated with a given maximum ground acceleration A_{max}, usually adopted in the earthquake resistant design practice as an input motion intensity parameter with respect to firm ground condition, then to an assumed A_{max} at the outcrops, say for example 0.3g, would nearly correspond σ_A=41.1cm/s2. The relation A_{max}-σ_A, in case of input motion intensity 0.3g for example, is established in the following way: knowing first that for an excitation function of the form of Eq. (1), $\omega_c=0.886s^{-1}$, $\omega_m=47.04s^{-1}$ and under the assumption for a 20sec input-output time duration the acceleration peak factor p_A [Davenport, 1964] results to be 3.58, σ_A at the engineering bedrock will become $\sigma_A = A_{max}/(2\times p_A) = (0.3\times 981)/(2\times 3.58) = 41.1 cm/s^2$.

STOCHASTICALLY BASED PREDICTION OF SITE SPECIFIC SPECTRAL RESPONSE

In case when input motion function at the engineering bedrock is defined and the spectral response at a given site is to be evaluated, the estimation of soil deposit transfer function is needed. In this study, a new formulation for the evaluation of site-specific response estimates based on the lumped mass model is conceived. On the basis of this model, an approximate estimation for the input motion intensity dependent complex transfer function of a given soil deposit can be obtained by making use of the following expression [Myslimaj and Matsushima, 1997]

$$|H_{FF}(\omega,\sigma_A)| \cong 2 \cdot \left| \sum_{j=1}^{m} \frac{1+2\cdot {}_g h_j(\sigma_A)\cdot[\omega/{}_g\omega_j(\sigma_A)]\cdot i}{1-[\omega/{}_g\omega_j(\sigma_A)]^2 + 2\cdot {}_g h_j(\sigma_A)\cdot[\omega/{}_g\omega_j(\sigma_A)]\cdot i} \cdot \psi_j \right| \quad ; \quad i=\sqrt{-1} \tag{3}$$

where ${}_gh_j(\sigma_A)$, ${}_g\omega_j(\sigma_A)$ and ψ_j represent respectively the total damping coefficient, the angular frequency and the free field participation function of the j-th mode of vibration. The evaluation of ${}_gh_j(\sigma_A)$, ${}_g\omega_j(\sigma_A)$ and ψ_j can be satisfactorily done by making use of the following closed form equations [Myslimaj and Matsushima, 1997; Myslimaj, 1998] :

$${}_gh_j(\sigma_A) = c\cdot(\sigma_A/g)^d + \frac{{}_g\omega_{1,0}}{{}_g\omega_{j,0}} \cdot \frac{2}{\pi} \cdot \frac{\rho_e V_e}{\rho_B V_B} \cdot \frac{1}{1+a(\sigma_A/g)^b} + \frac{{}_g\omega_{j,0}}{{}_g\omega_{1,0}} \cdot {}_g h_{v,0} \cdot \left[1+a(\sigma_A/g)^b\right] \tag{4}$$

$${}_g\omega_j(\sigma_A) = (2j-1)\cdot \frac{1}{1+a(\sigma_A/g)^b} \cdot \frac{\pi V_e}{2H} \tag{5}$$

$$\psi_j = (-1)^{j+1} \cdot \frac{1}{2j-1} \cdot \frac{4}{\pi} \tag{6}$$

where Ve is average shear wave velocity of the soil deposit, obtained by Ve= ΣViHi / ΣHi (here, Vi and Hi are the shear wave velocity and thickness of i-th layer respectively), ρe is the average mass density evaluated in the same way as Ve, ρB is the mass density of the base layer, VB is the shear wave velocity of the base layer and ghv,0 is initial viscous damping coefficient. gωj,0 is the linear angular frequency of the j-th mode of vibration, whose estimate can be obtained by (2j-1)·(πVe/2H) (here, H=ΣHi is soil deposit depth). a, b, c and d are non-dimensional constants determined in such a manner that respective numerical data mostly agree with the proposed models. Their values were found to be a=5.85, b=0.943, c=0.661 and d=0.772 [Myslimaj, 1998].

Having now defined the free field transfer function parameters through the closed form equations described above, one can easily estimate the acceleration power spectral-density function at the free surface SA,FF(ω,σA) as follows:

$$S_{A,FF}(\omega,\sigma_A) = \left|H_{FF}(\omega,\sigma_A)\right|^2 \cdot S_A(\omega,\sigma_A) \tag{7}$$

where SA(ω,σA) is the acceleration power spectral-density function of input motion at the engineering bedrock for a given level of σA.

Free field pseudo-velocity response spectrum pSV(T,h,σA) is obtained by the following approximate formula [Rosenblueth and Bustamante, 1962]:

$${}_pS_V(T,h,\sigma_A) = \sqrt{2\pi t_d S_{A,FF}(\omega,\sigma_A)} \cdot \sqrt{\frac{1-e^{-4\pi h\zeta}}{4\pi h\zeta} \cdot \left[0.424 + \ln(4\pi h\zeta + 1.78)\right]} \tag{8}$$

where T is the period in the response spectrum (=2π/ω), td is the input output time duration, ζ is a non-dimensional parameter (=td/T) and h is damping coefficient in the response spectrum.

Based on the equation (8) the expected pseudo-acceleration response spectrum, pSA(T,h,σA), can be easily obtained by making use of the following relation :

$${}_pS_A(T,h,\sigma_A) = \frac{2\pi}{T} \cdot {}_pS_V(T,h,\sigma_A) \tag{9}$$

STOCHASTIC SIMULATION OF SITE SPECIFIC SPECTRAL AMPLIFICATION

Based on the stochastic model described in the previous section, an approximate estimation for the site-specific spectral amplification (SSSA) accounting for the intensity of input motion can be obtained by making use of the following relation

$$SSSA(T,V_e,\sigma_A) \cong \frac{{}_pS_v(T,h=0,V_e,\sigma_A)\ \big|_{soil-free\ \ field}}{{}_pS_v(T,h=0,V_B,\sigma_A)\ \big|_{outcropping\ \ rock}} \tag{10}$$

The same relation has been used by Borcherdt [Borcherdt, 1994] for estimating short-period (0.1~0.5sec) and mid-period (0.4~2.0sec) average spectral amplification factors implied by the Loma Prieta Earthquake Strong Motion Data. These factors were derived from input ground-motion levels on firm to hard rock near 0.1g and represent frequency-based estimated averages over the corresponding frequency bands : (2~10Hz) and (0.5~ 2.5Hz). For higher levels they were inferred from numerical modeling results. Summarized results of his study for short- and mid-period spectral amplification factors, Fa and Fv, with respect to reference ground condition firm to hard rock (VB=1050m/s) for the local site conditions characterized in terms of average shear wave velocity Ve(m/s) to a depth of 30m, are shown in Table 1. Three representative values of Ve shown in this table, 540, 290 and 150m/s, correspond to site class II (SC-II), site class III (SC-III) and site class IV (SC-IV), respectively.

Table 1: Site-dependent amplification factors Fa and Fv [Borcherdt, 1994]

Amax	Fa			Fv		
	Ve=540	Ve=290	Ve=150	Ve=540	Ve=290	Ve=150
0.1	1.3	1.6	2.0	1.5	2.3	3.5
0.2	1.2	1.4	1.6	1.5	2.2	3.2
0.3	1.1	1.1	1.2	1.4	2.0	2.8
0.4	1.0	0.9	0.9	1.4	1.8	2.4

values of Fa and Fv adopted in the current BSSC seismic design provisions of 1995 [FEMA, 1995]. The short- and mid-period amplification factors shown in Table 1 provided the basis for the consensus

In order to see at what level of agreement are between them short- and mid-period spectral amplification factors predicted by the present study and those one shown in Table 1, a comparative analysis was performed, using H=30m, Ve=150, 290 and 540m/s with respective ρe=1.7, 1.75 and 1.8t/m3, ghv,0=0, VB=1050m/s, ρ B=2.0t/m3, td=20sec and σA=13.7; 27.4; 41.1; 54.8cm/s2 for the corresponding Amax=0.1; 0.2; 0.3; 0.4g of Table 1.

Comparative Analysis Results

Period-dependent spectral amplification curves, simulated for different soil conditions and intensity of shaking 0.1g and 0.3g, are shown as an examples in figures 1(a) and 1(b), respectively. These curves show that, in general, horizontal spectral amplification increases with decreasing average shear wave velocity Ve. Furthermore, the increase in amplification with decreasing Ve is distinctly less for short-period motion than for mid-period motion. This important observation suggests that site response can best be characterized by two factors, one for the short-period component of motion and one for the other period bands. This important result is most apparent for sites underlain by soft soils. It implies that average horizontal response characteristics at the sites can be summarized by amplification factors depending on the average shear wave velocity Ve.

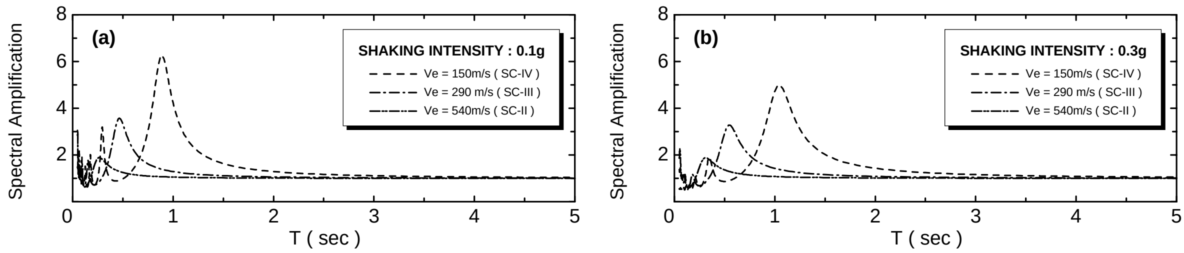

Figure 1: Site-specific spectral amplification curves for Amax=0.1g (a) and Amax=0.3g (b)

Short- and mid-period average spectral amplification factors, Fa and Fv, for three different local site conditions characterized in terms of average shear wave velocity Ve, are shown compared with those derived independently by Borcherdt [Borcherdt, 1994] in figures 2 through 4.

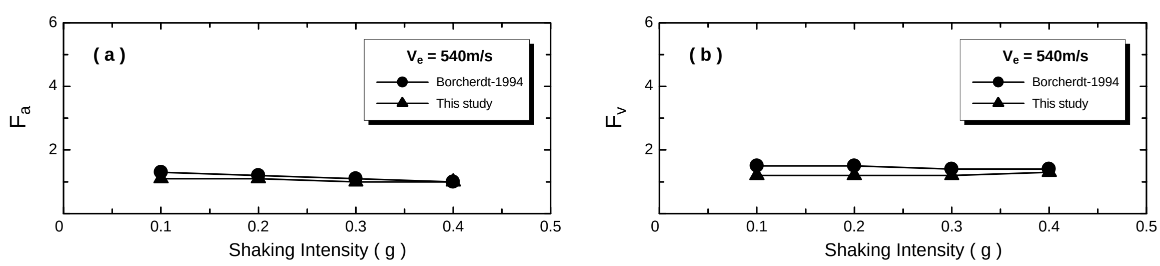

Figure 2: Fa (a) and Fv (b) amplification factors for Ve=540m/s

The degree of agreement between the results of the two independent studies is certainly noteworthy, particularly for Ve=540m/s and 290m/s. For the case of Ve=150m/s(Figure 4), average amplification factors derived by Borcherdt [Borcherdt, 1994] result to be larger than those predicted by this study. Most likely, this is due to the inadequacy of the assumed regression model to fairly fit the average amplification factors inferred from the strong-motion recordings on soft soil sites [Borcherdt, 1994], whose Ve lie within the interval 115~196m/s. In addition, it is interesting to notice here that for higher levels of the intensity of shaking results of both studies come closer. Most probably, this is related to the fact that amplification factors for higher shaking intensities were inferred from numerical modeling results.

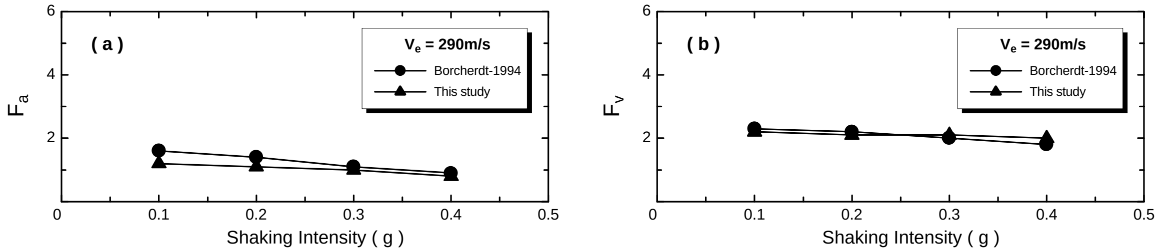

Figure 3: Fa (a) and Fv (b) amplification factors for Ve=290m/s

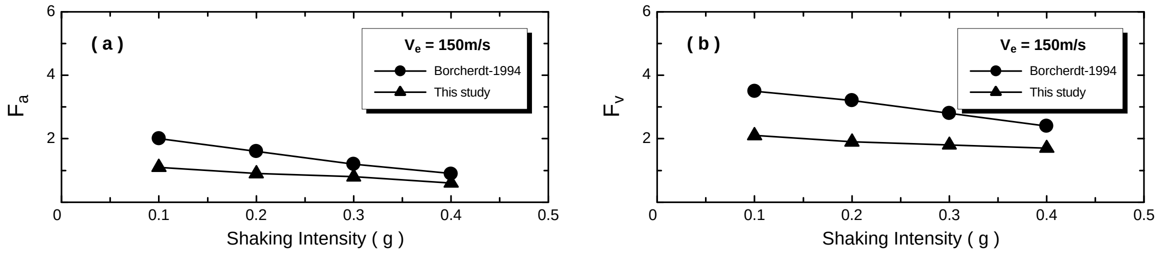

Figure 4: Fa (a) and Fv (b) amplification factors for Ve=150m/s

STOCHASTICI SIMULATION OF DESIGN RESPONSE SPECTRA ADOPTED IN BUILDING STANDARD OF JAPAN

In this paragraph, an attempt to simulate stochastically the design response spectra adopted in Building Standard Law of Japan approved in 1981(BSLJ-1981) [BCJ, 1981] is done, with the goal of examining the simulation capabilities of the proposed stochastic model through the comparison with already in use design approaches. Even though in seismic design load provisions of BSLJ-1981 it is not directly or clearly stated the term acceleration design spectrum, the non-dimensional period-dependent dynamic response coefficient Rt(T) implies that. Most likely, this is related to the fact that seismic load estimation procedure of BSLJ-1981 is based on base shear coefficient spectrum concept, which for a single-degree-of-freedom system could be simply presented as :

$$C(T) = Z \cdot R_t(T) \cdot C_o \tag{11}$$

where Z is the seismic zoning coefficient varying from 0.7 to 1.0, based on the seismic activity of the area where the prospective construction site is located, and Co is the standard base shear coefficient whose values for two earthquake-resistant design methods adopted in the Japanese seismic design practice, that is allowable stress design method and ultimate resistant force design method, are 0.2 and 1.0, respectively. If these two values of Co are to be correlated with peak ground motion parameters widely adopted in the seismic design practice, then to Co=0.2 and Co=1.0 would nearly correspond Amax=80~100cm/s2 and Amax=400~500cm/s2 [BCJ, 1981]. Extending further the above mentioned correspondence in a relation between Co and σA, we can finally relate Co=0.2 to σA=16cm/s2 and Co=1.0 to σA=80cm/s2 [Myslimaj, 1998]. It is important to notice here that above mentioned Co-Amax correspondence holds under the condition for Z and Rt equal to 1.0.

Regarding C(T) given by equation (11) as SA(T,h=5%)/g the following approximate relation between Rt(T) and 5% damped acceleration response spectrum can be established:

$$S_A(T, h = 5\%) \cong C_o \cdot R_t(T) \cdot g \tag{12}$$

Period-dependent dynamic response coefficient Rt(T) appearing in the equations (11) and (12) is also dependent on the site conditions which are classified in three classes [BCJ, 1981], based on the ground period Tg : Site

Class I Tg0.2s; Site Class II 0.2sTg0.75s and Site Class III 0.75s Tg. Since site classification is based on ground period Tg, which on average can be evaluated by 4H/Ve, it is of particular importance to examine first in what relation are between them the two factors affecting Tg estimation : soil deposit depth H and its average shear wave velocity Ve. In order to do that, the relation H-Ve for a sample of 30 soil profiles, recently analyzed for the purpose of soil deposit amplification characteristics investigation [Myslimaj, 1998], is presented in figure 5. No trend of correlation can be seen from this plot. Lack of correlation between H and Ve, characterized by an all over scattering of data (mostly located within the rectangle defined by the lines Ve=100m/s; Ve=300m/s and H=10m; H=50m), suggests that in the estimation of Tg, H and Ve should be treated as two independent variables, having probability density functions of the forms shown in figures 6 and 7, respectively. Moreover, considering the fact that Tg=4H/Ve, one can expect that the probability density function for Tg be of the form shown in figure 8, where two vertical doted lines at 0.2s and 0.75s mark the boundaries between three different site classes.

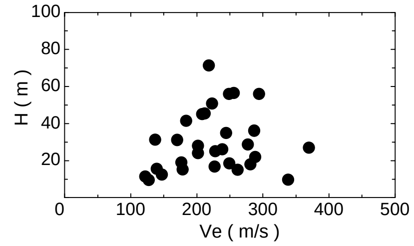

Figure 5: H - Ve relation for the sample considered

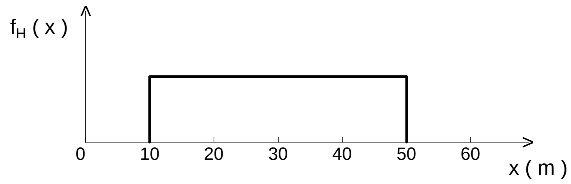

Figure 6: Probability density function for H

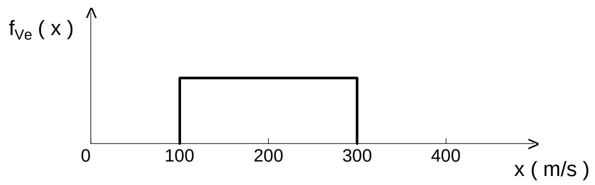

Figure 7: Probability density function for Ve

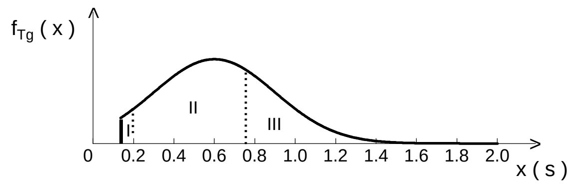

Figure 8: Probability density function for Tg

On this basis, average acceleration response spectrum, for each soil class and for a given input motion intensity level σA, SA,C(T,h,σA), can be obtained by :

$$S_{A,C}(T,h,\sigma_A) = \int_C {}_pS_A(T,h,\sigma_A,T_g)\, f_{T_g}(x)dx = \iint_C {}_pS_A(T,h,\sigma_A,H,V_e) f_H(x) f_{V_e}(y)dxdy \tag{13}$$

where pSA(T,h,σA,Tg) is the expected free field pseudo acceleration response spectrum of a soil deposit of fundamental period Tg, estimated on the basis of formula (9). The subscript C in the above written equation stands for the site class considered. Corresponding variance and coefficient of variation can be evaluated by making use of the following relations:

$$Var(S_{A,C}) = \int_C \left[{}_pS_A(T,h,\sigma_A,T_g) - S_{A,C}(T,h,\sigma_A) \right]^2 f_{T_g}(x)dx \tag{14}$$

$$C.O.V. = \sqrt{Var(S_{A,C})} \Big/ S_{A,C} \tag{15}$$

Based on the above described model, a sample of 1681 soil profiles consisting of 66 sites of Class I, 1062 sites of Class II and 553 sites of Class III was analyzed, with the goal of simulating the acceleration response spectra adopted in BSLJ-1981. The total number of soil profiles appeared above results from the discretization of each of the continuous probability density functions shown in figures 6 and 7 in 41 discrete variables, gradually increasing from the minimum value up to the maximum one with an increment of 1m for H and 5m/s for Ve. The sample covers a ground period range Tg=0.13s~2.0s (see figure 8). The values of other parameters needed for the stochastic simulation of input motion intensity dependent site-specific response spectra were assumed as: VB=400m/s; ρB=2.0t/m3; ρe=1.75t/m3; ghv,0=0; td=20sec and h=5%. For comparison reasons with acceleration design response spectra adopted in BSLJ-1981, two levels of acceleration standard deviation at the engineering bedrock level were assumed: σA=16cm/s2 and σA=80cm/s2.

RESULTS OF ANALYSIS

Results of this comparative analysis, for each site class and input motion intensity level, are shown in figures 9 and 10. The most interesting thing which can be noticed from these results is that stochastically predicted response spectra are consistent with those adopted in the seismic design practice of Japan, even if both of them have resulted from quite different methods of analysis. In particular, satisfactory degree of agreement is noticed for Site Class II, regardless of input motion intensity level. The sharp peak appearing in the simulated acceleration response spectra of Site Class I (figures 9(a) and 10(a)) is related to the fact that for this site class the number of soil profiles analyzed (66) is far less than those of Site Class II (1062) and Site Class III (553), adding also that Site Class I soil profiles cover a quite narrow period band (0.13s~0.20s) compared with period bands covered by Site Class II soil profiles (0.20s~0.75s) and Site Class III soil profiles (0.75s~2.0s).

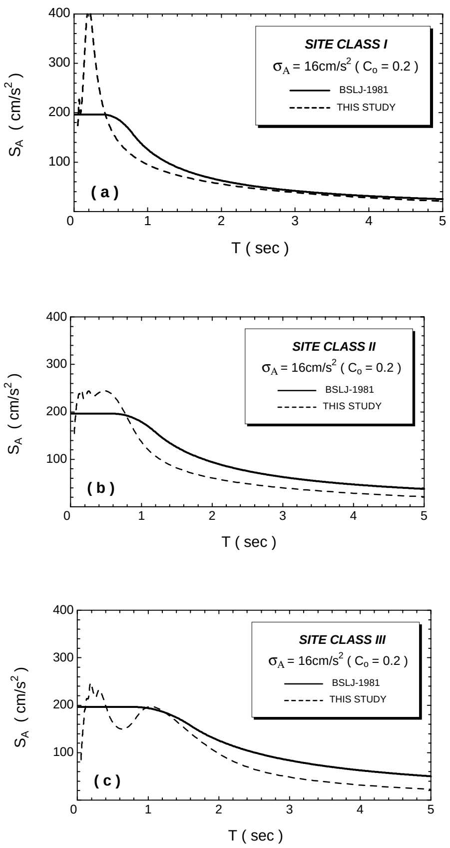

Figure 9: BSLJ-1981 design response spectra and stochastically simulated ones for Co=0.2

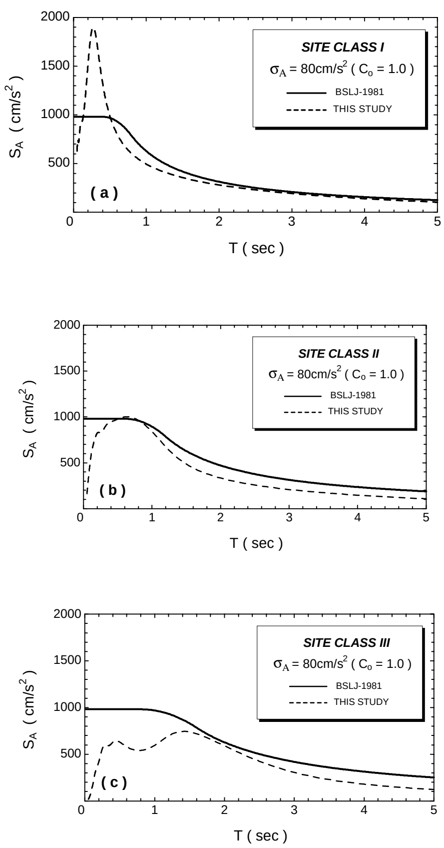

Figure 10: BSLJ-1981 design response spectra and stochastically simulated ones for Co=1.0

(a) Site Class I, (b) Site Class II and (c) Site Class III

While there is a slight and negligible reduction of acceleration spectral response at short-period range for Site Class I (Hard) and Site Class II (Intermediate) at relatively high input motion level Co=1.0 or σA=80cm/s2 (compare figures 10(a), (b) with figures 9(a),(b)), clearly noticed lower acceleration spectral response than BSLJ-1981 has resulted for the Site Class III (Soft) at the same period range (compare figure 10(c) with figure 9(c)). Most likely, this is related to the fact that the stochastically simulated response spectra have taken into account the nonlinear stress-strain behavior in the soil as the input motion intensity increases. This effect seems to have strongly affected the average acceleration spectral amplification for soft soil sites (Site Class III), which tends to decrease as the input acceleration increases [Idriss, 1990; Borcherdt, 1994]. Comparing results shown in figure 9(c) with those one shown in figure 10(c), we can conclude that the anticipated nonlinear effects in soft soil sites are (1) reduction of spectral amplification at short period range and (2) the shift of peak spectral response to the longer period range, both occurring as the input motion intensity increases from σA=16cm/s2 to σA=80cm/s2.

The differences between stochastically simulated site-dependent response spectra accounting for input motion intensity are clearly reflected also in the variability of C.O.V. for each Site Class, shown in figures 11 and 12. In general, an all over less than 0.4 C.O.V. has resulted for both input motion intensity levels. Here again, slight change of C.O.V. for Site Class I and II has resulted for two different input motion intensities, compared to that one of Site Class III where the widening of high values portion of C.O.V. for the higher level of input motion intensity (Figure 12) is clearly noticed.

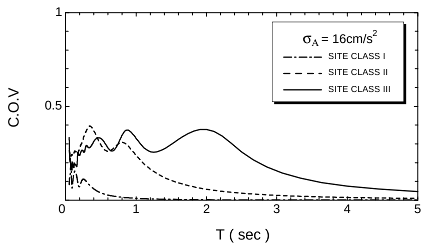

Figure 11: Period-dependent C.O.V for different site classes and σA = 16cm/s2

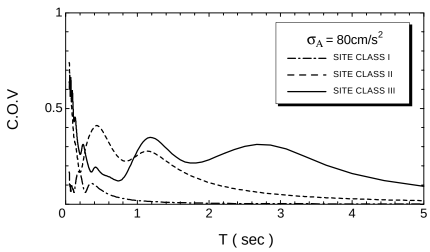

Figure 12: Period-dependent C.O.V for different site classes and σA = 80cm/s2

CONCLUSIONS

In absence of recorded strong ground motions at various intensity levels required for a proper earthquake resistant design, theoretical earthquake input motion descriptions have proven to be effective in predicting ground motion parameters at a given site, in an approximate but acceptable form for practical design purposes. In this study a newly formulated stochastic model for predicting site-specific spectral response was presented. Model's performance was tested through the comparison with already in use design approaches, such as BSSC Seismic Design Provisions of 1995 and Building Standard Law of Japan of 1981. The satisfactory degree of agreement between the results of this study and those obtained through other quite different design approaches gives confidence that the proposed stochastic model can be used for estimating input motion intensity dependent site-specific spectral amplification factors for design purposes and improving site-dependent building-code provisions. It can be of further use in performance-based seismic design procedures, where the prediction of prospective site-specific design spectra accounting for the nonlinear behavior of surface soil layers is needed.

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