Analytical evaluation of site-specific response spectrum

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ABSTRACT

An approach for nonlinear response analysis of sites in the eastern United States is presented to evaluate the characteristics of earthquake ground motions. Since strong-motion data in the eastern United States are very scarce, we use a seismologic model to generate a synthetic acceleration time history at the base of the soil profile. The site response analysis is performed by using the MASH computer program, in which the hysteretic models for sand and clay proposed by Hwang and Lee (1990) is used. The effect of soft-rock on the site-specific response spectrum is discussed.

INTRODUCTION

Estimating the characteristics of ground motions induced by large earthquakes occurring in the eastern United States is quite challenging because only a few strong-motion data were recorded. In addition, soil conditions at a site have significant effects on the characteristics of earthquake ground motions and corresponding response spectrum. Earthquake motions at the bedrock level can be drastically modified in frequency contents and amplitude as seismic waves are transmitted through a soil deposit. Using the site of the Sheahan pumping station, Memphis, Tennessee, as an example, we propose an approach to evaluate the site-specific response spectrum for design of structures.

DYNAMIC SOIL MODEL

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The existing boring log of the Sheahan pumping station terminates at 52 ft (16 m). The soil profile is extended from 52 ft (16 m) to 200 ft (61 m) using a water-well log in the vicinity of the Sheahan pumping station. The soil at this depth is very stiff and thus the base of the soil profile is chosen at this level. The bedrock in the Memphis area is about 3000 ft (909 m) below the ground surface. The material between 200 ft (61 m) and 3000 ft (909 m) is denoted as soft-rock and its effect on the ground motion is included in the input synthetic earthquake time history. The soil profile of the Sheahan pumping station is shown in Fig. 1.

The site response analysis is performed using the MASH computer program (Martin and Seed 1978). The dynamic soil model in the MASH program consists of a horizontally multi-layered soil profile with a fixed base. Static and dynamic properties of soil layers and location of water table are needed to define a dynamic soil model. In this study, static soil properties are either taken from available existing boring logs in the Memphis area or estimated based on empirical correlations. Soil exhibits pronounced nonlinear behavior under cyclic loadings. The secant shear modulus G is strain-dependent and decreases with increasing shear strain levels γ. In the MASH program, the secant shear modulus is expressed as

$$\frac{G}{G_o} = 1 - \left[\frac{[\gamma/\gamma_o]^{2B}}{1 + [\gamma/\gamma_o]^{2B}}\right]^A \tag{1}$$

where G_o is the low-strain shear modulus; γ_o is the reference strain; and the parameters A and B describe the shape of the normalized shear modulus reduction curve. In the MASH program, G_o in psf for sand is estimated from the following empirical equation:

$$G_o = 61000[1 + 0.01\,(D_r - 75)]\,(\bar{\sigma})^{1/2} \tag{2}$$

where D_r is the relative density in percentage and $\bar{\sigma}$ is the average effective confining pressure in psf. The reference strain γ_o is expressed as τ_{max} / G_o. τ_{max} is the maximum shear stress under dynamic loadings and is computed using the formula suggested by Hardin and Drnevich (1972). The parameters A and B were determined by Hwang and Lee (1990) as 0.941 and 0.441, respectively.

Several studies have demonstrated that the plasticity index PI is the most dominant factor affecting the shape of the shear modulus reduction curve for clay. In general, the shear modulus exhibits a smaller reduction with increasing plasticity index at the same shear strain level. The low-strain shear modulus G_o for clay in the MASH program is computed as $G_o = 2500\,S_u$, where S_u is the undrained shear strength of clay and is taken as one half the unconfined compressive strength q_u. In this study, τ_{max} is taken as S_u and G_o is taken as $2500\,S_u$; thus the reference strain γ_o is equal to 0.0004. Sun et al. (1988) suggested the shear modulus reduction curves for clay corresponding to different ranges of plasticity indices. Using nonlinear regression analysis, Hwang and Lee (1990) determined the values of parameters A and B for these curves.

INPUT EARTHQUAKE MOTION

Seismic hazard in Memphis and Shelby County, Tennessee, is entirely dominated by the New Madrid seismic zone (NMSZ). In this study, a New Madrid earthquake of moment magnitude M = 7.5 is assumed to occur at Marked Tree, Arkansas, which is near the southern end of the NMSZ. The epicentral distance R from the source to the Sheahan pumping station is about 67 km. Since strong-motion data in the eastern United States are very scarce, a seismologic model is used to estimate the horizontal motions at the base of the soil profile. In this model, source mechanism, path attenuation and soft-rock effects are considered to establish ground motions primarily due to shear waves generated from a seismic source.

The Fourier amplitude spectrum $A(f)$ is formulated following the approach proposed by Boore and Atkinson (1987).

$$A(f) = C \times S(f) \times D(f) \times I(f) \times AF(f) \tag{3}$$

where C is a scaling factor; $S(f)$ is a source spectral function; $D(f)$ is a diminution function; $I(f)$ is a shape filter; and $AF(f)$ is an amplification factor. These factors except the amplification factor are explained in detail by Boore and Atkinson (1987). The values of the parameters that defines these factors are summarized in Table 1.

Table 1. Summary of seismic parameters

	SYMBOL	VALUE
Moment Magnitude	M	7.5
Epicentral Distance	R	67 km
Focal Depth	h	10 km
Radiation Pattern	$<R_{\theta\phi}>$	0.55
Horizontal Component	V	0.71
Shear Wave Velocity	β	3.5 km/sec
Source Rock Density	ρ	2.7 gm/cm^3
Quality Factor	$Q(f)$	$1500f^{0.4}$
Stress Parameter	$\Delta\sigma$	150 bars
Cut-Off Frequency	f_m	30 Hz
Strong-Motion Duration	T	16 sec

The amplification factor $AF(f)$ accounts for the soft-rock effects resulting from the decreasing shear-wave velocities in the soft-rock layers. $AF(f)$ can be calculated as (Boore 1987):

$$AF(f) = \sqrt{\frac{\beta}{\beta_r}} \tag{4}$$

where β is the shear-wave velocity at the source and β_r is the effective shear-wave velocity. The method to calculate β_r proposed by Boore (1987) requires that the shear-wave velocity and layer thickness of all soft-rock layers be known. Based on the shear-wave velocity and thickness of the soft-rock layers in the Memphis area as suggested by Jacob, the amplification factors are calculated and shown in Table 2.

Table 2. Calculation of the amplification factor

H_i (m)	ΣH_i (m)	β_i (m/s)	T_n (sec)	f_n (Hz)	β_r (m/s)	AF
9	9	400	0.02225	11.11	400.00	2.96
13	22	600	0.04392	5.69	500.95	2.64
17	39	1000	0.06092	4.10	640.22	2.34
100	139	950	0.16618	1.50	836.44	2.04
300	439	1100	0.43891	.57	1000.89	1.87
200	639	1400	0.58176	.43	1129.91	1.76
200	839	1700	0.69941	.36	1197.00	1.71
100	939	2000	0.74941	.33	1254.98	1.67
2000	2939	3000	1.41607	.18	2071.00	1.30
7061	10000	3500	3.43350	.07	2892.60	1.10

An earthquake accelerogram generally shows a build-up segment followed by a strong-motion segment and then a decay segment. The frequency content of earthquake accelerograms is found to be approximately constant during the strong-motion segment. Thus, the strong-motion segment of an acceleration time history is considered as a stationary random process and the one-sided power spectrum $S_a(f)$ can be derived from the Fourier amplitude spectrum.

$$S_a(f) = \frac{2}{T_e} |A(f)|^2 \tag{5}$$

where $A(f)$ is the Fourier amplitude spectrum in Eq. 3. T_e is the strong-motion duration and is equal to the source duration which is the reciprocal of the corner frequency f_0, that is, $T_e = 1/f_0$. In this study, the synthetic time histories are generated using the method proposed by Shinozuka (1974). Given the power spectrum, the stationary acceleration time histories $a_s(t)$ can be generated as follows:

$$a_s(t) = \sqrt{2} \sum_{k=1}^{N} \sqrt{S_a(\omega_k)\Delta\omega}\, \cos(2\pi\omega_k t + \phi_k) \tag{6}$$

where $S_a(\omega_k)$ = one-sided earthquake power spectrum; N = number of frequency intervals; $\Delta\omega$ = frequency increment; $\omega_k = k\,\Delta\omega$; ϕ_k = random phase angles uniformly distributed between 0 and 2π. The nonstationary acceleration time histories a(t) can then be obtained from the multiplication of an envelope function w(t) to the stationary time history. The envelope function w(t) used in this study is composed of three segments: (1) a parabolically increased segment simulating the initial-rise part of the accelerogram and its duration is chosen as one fifth of T_e, (2) a constant segment representing the strong-motion portion of an earthquake excitation and has a duration equal to T_e, and (3) a linearly decayed segment extending four fifths of T_e. Thus, the total duration is $2T_e$. Real earthquake records are commonly observed with long coda durations; however, the coda durations are considered unimportant in most engineering applications.

SITE-SPECIFIC RESPONSE SPECTRA

Using the MASH program, the acceleration time history and corresponding response spectrum with 5% damping ratio at the ground surface are computed based on the earthquake-site model described above. The response spectra (denoted as Case 1) at the ground surface and at the base of the soil profile are shown in Fig. 2. The frequency contents of the base motions have been drastically modified. The spectral values of ground acceleration are significantly higher than those of base accelerations between the period of 0.2 to 1.1 seconds. On the other hand, the high frequency contents of base acceleration, say less than 0.2 second, are significantly reduced as shear waves transmit through the soil deposit.

SOFT-ROCK EFFECT

The approach described above requires that the properties of all the soil and soft-rock layers between the ground surface and the bedrock be known. For a site with a great depth to the bedrock such as the site of the Sheahan pumping station used in this study, the properties of soft-rock layers may be obtained using the reflection analysis or other techniques. However, the results cannot be determined precisely. Assuming we do not have soft-rock data at a site, we still can establish the site model using the upper soil layers as described in this paper. However, the input earthquake time history is generated based on the bedrock data and including the free-surface effects, i.e., neglecting the existance of soft-rock and soil layers. Using this approach, the response spectum at the ground surface and the input response spectrum, denoted as Case 2, are also shown in Fig. 2. It can be seen that even though the input response spectra have significant difference, the resulting ground response spectra are very similar. Thus, for engineering application, if we perform a nonlinear site response analysis with reasonable deep soil profile, we can obtain a fairly accurate response spectrum for the design of structures.

CONCLUSIONS

This paper presents an approach for the nonlinear response analysis of sites in the eastern United States to evaluate the characteristics of earthquake ground motions. The conclusions from this study are as follows:

1. The analytical approach described in this paper combines a seismologic model for input motion and a nonlinear site response analysis. This approach is a reasonable way to generate ground response spectrum for a site in the eastern United States, since strong-motion data are very scarce in this region.

2. The response spectrum at the ground surface is quite different from that at the base of the soil column because of the nonlinear behavior of soil under cyclic loadings. Thus, nonlinear site response analysis must be performed, when a site is subject to large earthquakes.

3. For a site with deep soil profile and several layers of soft rocks overlain the bedrock, the ground response spectrum can be approximately estimated by using the input base motions taken as the bedrock motion at the free surface.

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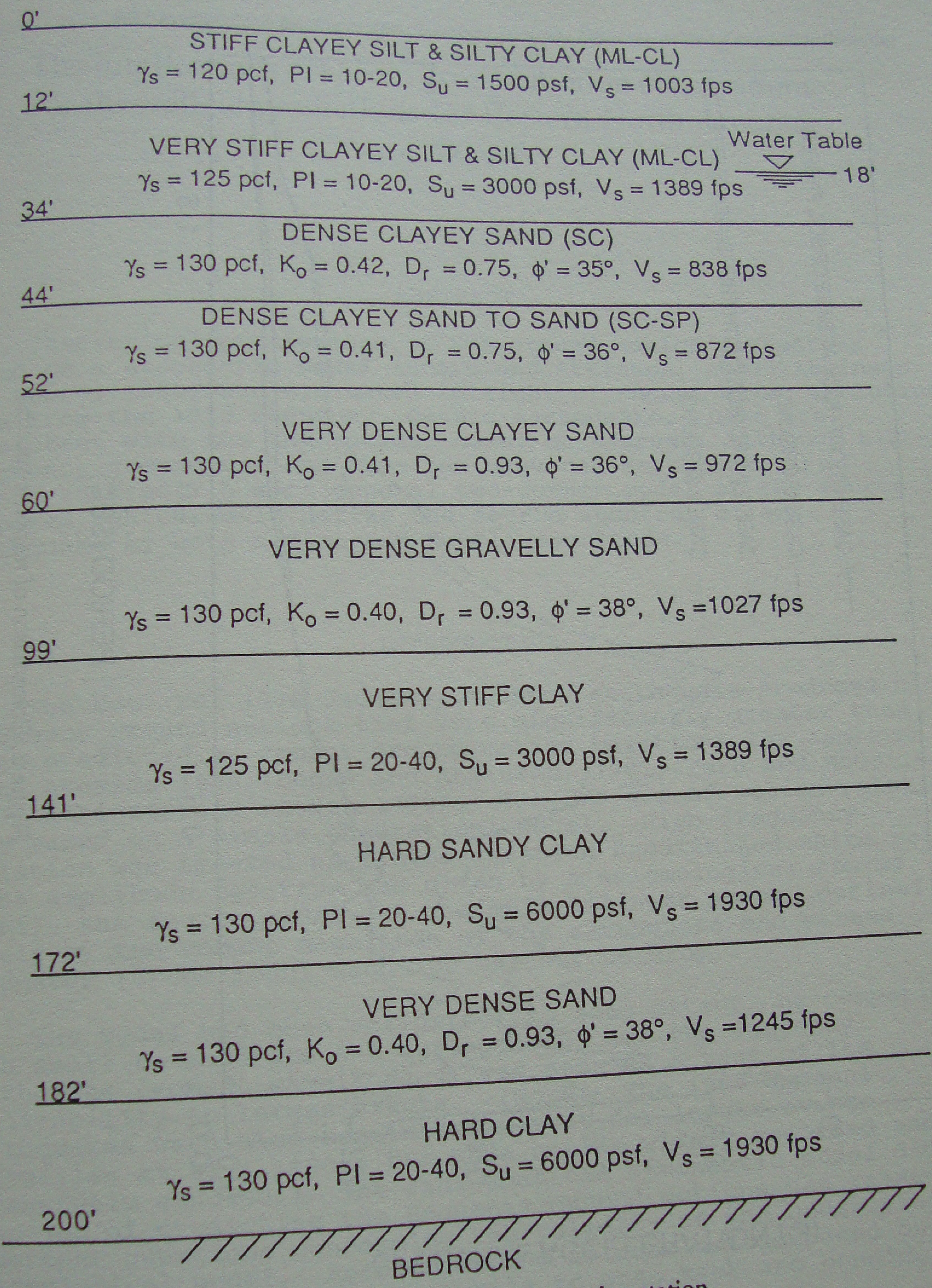

Figure 1. Soil profile for Sheahan pumping station

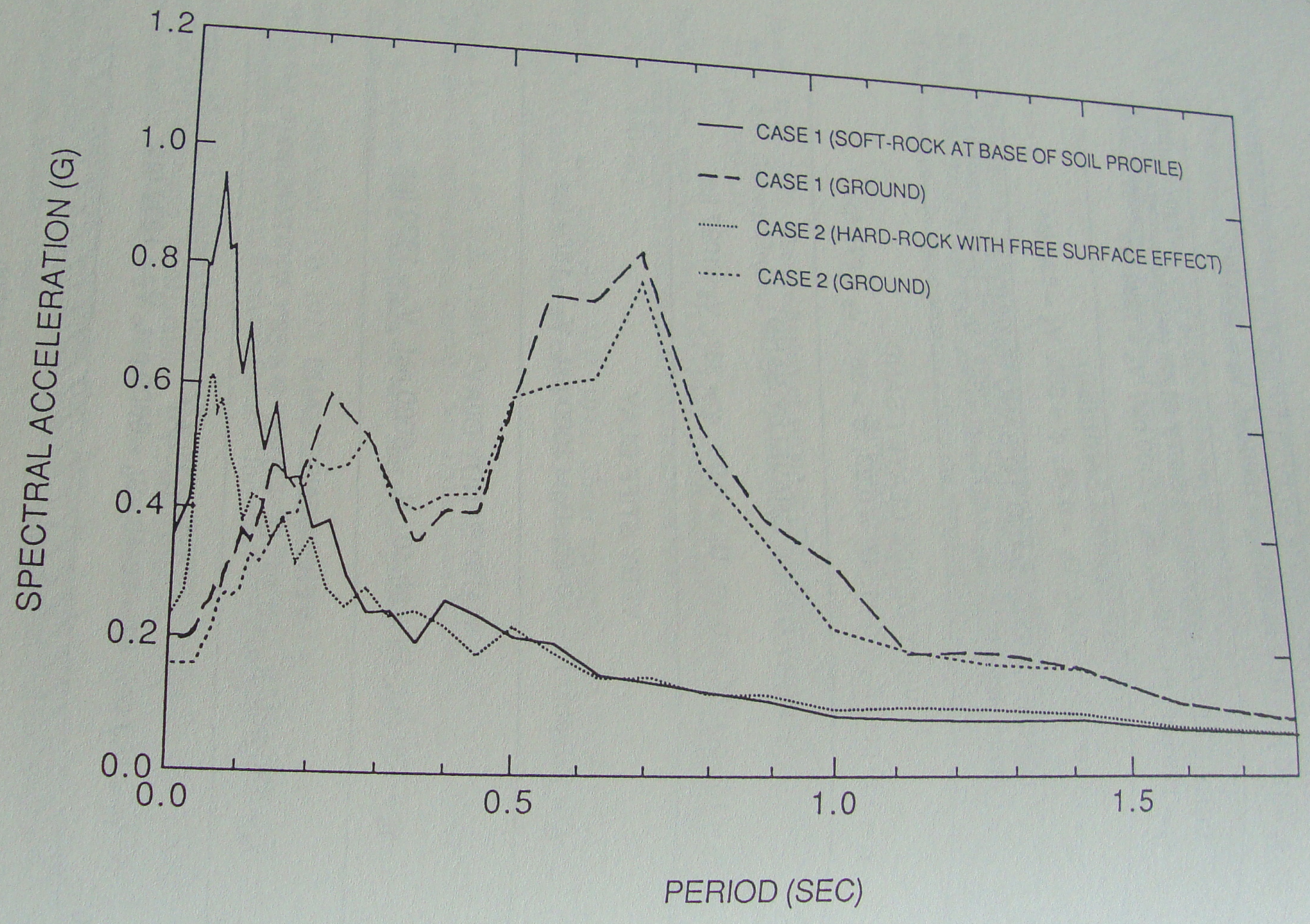

Figure 2. Comparison of response spectra