New Site Coefficients and Site Classification System Used in Recent Building Seismic Code Provisions

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Recent code provisions for buildings and other structures (1994 and 1997 *NEHRP Provisions*, *1997 UBC*) have adopted new site amplification factors and a new procedure for site classification. Two amplitude-dependent site amplification factors are specified: F_a for short periods and F_v for longer periods. Previous codes included only a long period factor S and did not provide for a short period amplification factor. The new site classification system is based on definitions of five site classes in terms of a representative average shear wave velocity to a depth of 30 m ($\bar{V}_s$). This definition permits sites to be classified unambiguously. When the shear wave velocity is not available, other soil properties such as standard penetration resistance or undrained shear strength can be used. The new site classes denoted by letters A - E, replace site classes in previous codes denoted by S1 - S4. Site classes A and B correspond to hard rock and rock, Site Class C corresponds to soft rock and very stiff / very dense soil, and Site Classes D and E correspond to stiff soil and soft soil. A sixth site class, F, is defined for soils requiring site-specific evaluations. Both F_a and F_v are functions of the site class, and also of the level of seismic hazard on rock, defined by parameters such as A_a and A_v (*1994 NEHRP Provisions*), S_s and S_1 (*1997 NEHRP Provisions*) or Z (*1997 UBC*). The values of F_a and F_v decrease as the seismic hazard on rock increases due to soil nonlinearity. The greatest impact of the new factors F_a and F_v as compared with the old S factors occurs in areas of low-to-medium seismic hazard. This paper summarizes the new site provisions, explains the basis for them, and discusses ongoing studies of site amplification in recent earthquakes that may influence future code developments.

INTRODUCTION

It has long been recognized that the effect of geologic and local soil conditions on ground motion characteristics should be considered in the seismic design of buildings and other

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structures. The *ATC 3* report of the Applied Technology Council, *Tentative Provision for the Development of Seismic Regulations for Buildings* (ATC 1978), that introduced this effect into U.S. seismic building codes provided for the use of three site categories different enough to warrant three distinct site coefficients (factors S for site profile types S1, S2, and S3). Between 1976 and 1994, the site categories and site coefficients S1, S2, and S3 were defined by descriptions that took into account both the stiffness and depth of soil at the site. These site categories and site coefficients were based primarily on several statistical studies of earthquake studies conducted by H. B. Seed and his coworkers (Seed et al. 1976a, b) and by Mohraz (1976). Experience from the September 1985 Mexico City earthquake (Seed et al. 1988, BSSC 1988) prompted the addition of a fourth site category and an associated, higher site coefficient for deep soft clay deposits (S4). These S factors were embedded in an approach in which each site category was associated with a different spectral shape, and the S factors only amplified the long period part of the spectrum. The experiences of both the 1985 Mexico City and 1989 Loma Prieta earthquakes, as well as other observations and studies, showed the effect of the level of shaking, and especially the fact that low levels of peak ground acceleration and associated low spectral levels at short periods can be significantly amplified at soft sites (Idriss 1990, 1991; see also Figure 1). While the level of rock shaking in the San Francisco Bay area during the 1989 Loma Prieta earthquake was lower than that used as a basis for the code provisions in high seismicity areas of California, it was comparable to that used for other, lower seismicity areas of the country. The efforts toward a seismic building code for New York City (Jacob 1990, 1994; New York City 1995), were the first to incorporate two key aspects that have now become part of the *NEHRP* (National Earthquake Hazards Reduction Program) *Recommended Provisions for the Development of Seismic Regulations for New Buildings and Other Structures* and the *Uniform Building Code (UBC)*: (1) higher values of soil site coefficients as appropriate for areas of lower shaking, and (2) the addition of a "hard rock" category to better reflect rock geologic conditions in the eastern United States.

The need for improvement in codifying site effects at the national level was discussed at a 1991 NCEER[1] workshop devoted to the subject (Whitman 1992). As a result a committee of nine members (the authors of this paper, chair M. Power) was formed during that workshop to study the situation and develop specific code recommendations. The committee collected information, guided related research, discussed the issues, and organized a site response workshop in Los Angeles in November 1992 (see Martin 1994). This workshop, attended by 65 invited geoscientists, geotechnical engineers, and structural engineers, developed the consensus recommendation on new site categories and site coefficients that were eventually incorporated into the 1994 and 1997 *NEHRP Provisions* and *1997 UBC* provisions.

BASIC CONSIDERATIONS

Figure 1 presents average horizontal elastic response spectra recorded on soft clay and rock sites in San Francisco and Oakland during the Loma Prieta earthquake. While the peak acceleration on rock was about 0.08 or 0.1g, it was amplified two to three times to 0.2g or

1. NCEER: National Center for Earthquake Engineering Research (now MCEER: Multidisciplinary Center for Earthquake Engineering Research).

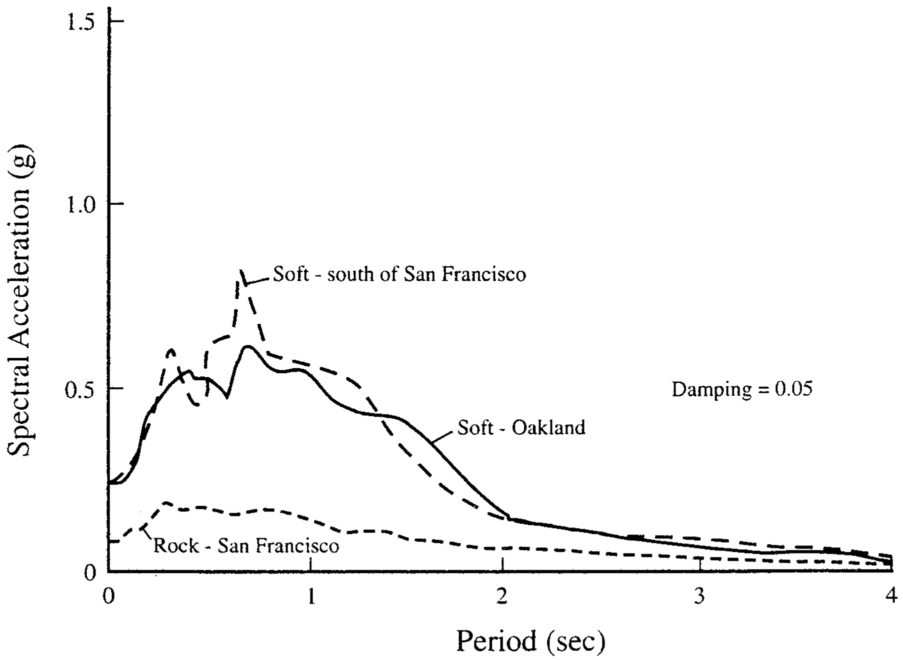

Figure 1. Average spectra recorded during 1989 Loma Prieta earthquake in San Francisco Bay area at rock sites and soft soil sites (modified after Housner 1990).

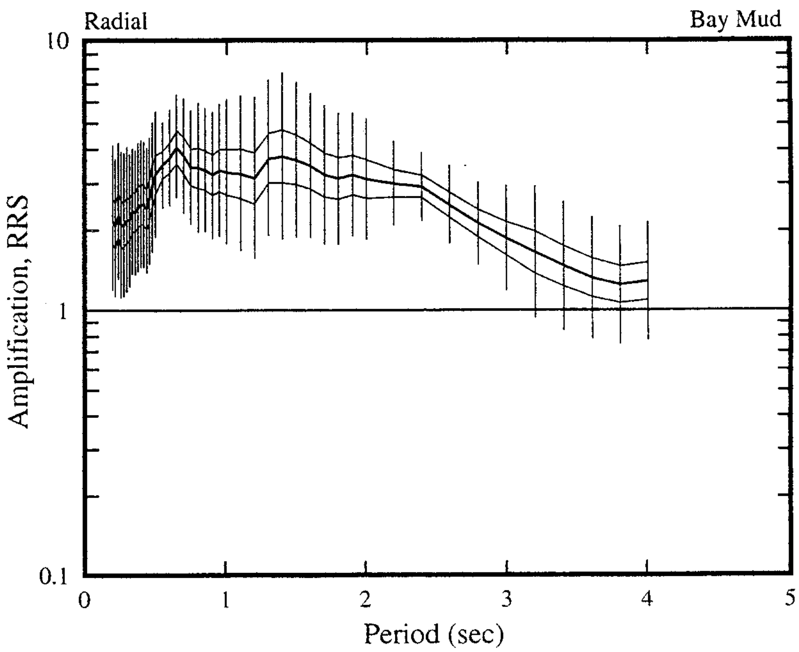

Figure 2. Calculation of average RRS curves for 5 percent damping from records of 1989 Loma Prieta earthquake on soft soil sites. The middle curve gives the geometric average ratio as function of the period. The top and bottom curves show the range from plus to minus one standard deviation of the average of the logarithms of the ratios. The vertical lines show the range from plus to minus standard deviation of the logarithms of the ratios (Joyner et al. 1994).

0.3g at the soft soil sites; the spectral ordinates at short periods ($T \approx 0.2$ or 0.3 seconds) were also amplified on average by factors of 2 or 3. Figure 2 shows the result of a statistical analysis of this amplification of response spectra between nearby rock and soil sites, also called Ratio of Response Spectra (RRS), versus period, for a number of soft sites in this earthquake (Joyner et al. 1994). Both Figures 1 and 2 show that at longer periods between 0.5 and 1.5 or 2 seconds the amplification is even greater than at short periods, with RRS now ranging from three to six times. A similar amplification behavior but with lower values of RRS was observed between stiff soil sites and rock sites (Housner 1990, Chang 1991, and Joyner et al. 1994). Figure 3 includes the profile of a representative, instrumented soft clay site south of San Francisco, about 70 km north of the epicenter, which provided one of the records included in Figures 1 and 2. At some of these soft sites, the maximum value of RRS, RRS_{max}, occurred at a long period seemingly related to the characteristics of the soil deposit. For the site of Figure 3, $RRS_{max} \approx 3.5$ at a period $T \approx 1.4$ sec. An even more extreme case of this type of amplification has been observed in the very soft clay deposits of Mexico City at periods of the order of 2 or 3 seconds, with RRS_{max} ranging from about 3 to 20. Fortunately, as discussed later herein, both the extreme softness of the soil as well as other characteristics of the Mexico City clay inducing these very high amplifications are rather unusual.

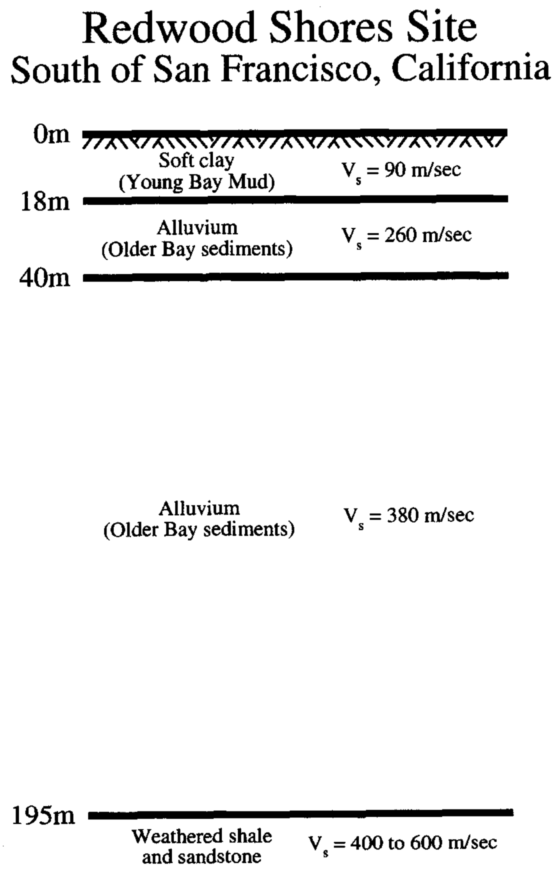

Figure 3. Soil profile at instrumented soft clay site that recorded the 1989 Loma Prieta earthquake (Chang 1991, Dobry 1991).

Useful insight on the factors controlling the value of RRS_{max} and the amplification phenomenon at soft sites revealed by Figures 1 through 3 is provided by the 1D amplification model of Figure 4 and the results in Figure 5 (Roesset 1977). Both the horizontal acceleration on rock at point B, a_B, and the corresponding soil acceleration at point A, a_A, are caused by vertically propagating, sinusoidal shear waves of frequency f (cps). Therefore, both a_A and a_B are amplitudes of sinusoidal accelerograms of frequency f. The amplification ratio a_A / a_B is a function of the ratio of frequencies $f / (V_s / 4h)$, of the soil material damping ratio β_s, and of the rock/soil impedance ratio, $I = \gamma_r V_r / \gamma_s V_s$. In Figure 4, γ_r, γ_s are the total unit weights and V_r, V_s are the shear wave velocities of rock and soil. The maximum amplification $(a_A / a_B)_{max}$ corresponding to resonance in shear of the soil layer, occurs at a frequency near $V_s / 4h$, where $V_s / 4h$ is the undamped fundamental frequency of the layer. This maximum amplification is approximately equal to:

$$\left[\frac{a_A}{a_B}\right]_{max} \approx \frac{1}{(1/I)+(\pi/2)\beta_s} \tag{1}$$

In Figure 5, $V_s / 4h = 1.88$ cps, $I = 6.7$, and for $\beta_s = 0.05$, $(a_A / a_B)_{max} \approx 4.4$. A plot such as Figure 5 provides the transfer function of the site, which is constant and independent of the rock motion for the assumed linear soil and vertically propagating shear waves. While the transfer function of a layered soil profile such as that of Figure 3 may be more complex than Figure 5, every soil profile on stiffer rock has a transfer function that is constant and independent of the rock motion, under the assumed linear soil and vertically propagating shear waves. The transfer function can be properly estimated by dividing Fourier Spectra of recorded horizontal accelerations on nearby soil and rock sites. On the other hand, plots of RRS, such as discussed before and plotted in Figure 2 for the Loma Prieta earthquake records, are more appropriate for engineering evaluations involving response spectra and for determination of site coefficients in seismic codes; however, these ratios are not independent of the rock motion. Fortunately, analyses and comparisons of actual soft clay sites on much

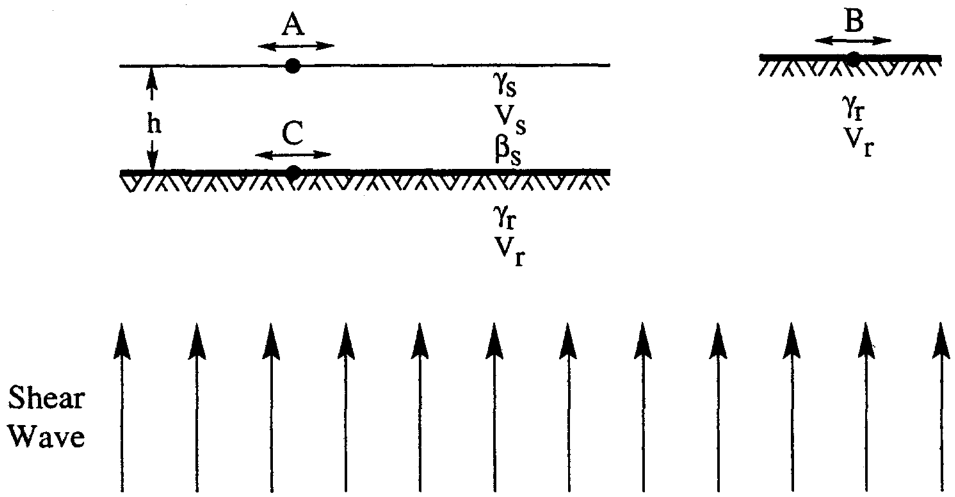

Figure 4. Uniform soil layer on elastic rock subjected to vertical shear wave.

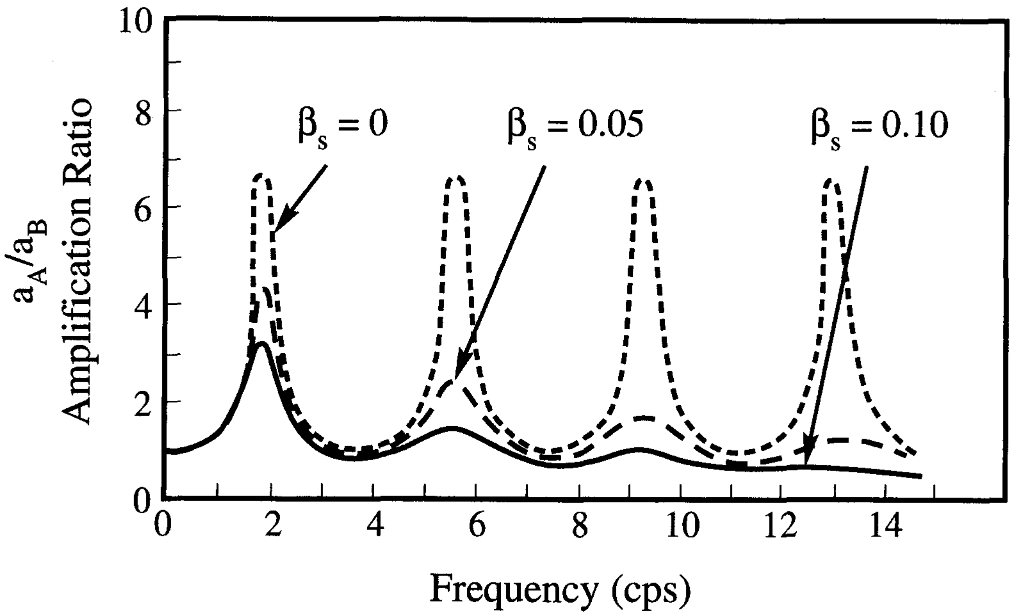

Figure 5. Amplification ratios soil/rock for h = 100 ft (30.5m), V_s / 4h = 1.88 cps, and I = 6.7 (Roesset 1977).

stiffer rock or soil, suggest that (1) both RRS_{max} and $(a_A / a_B)_{max}$ occur at about the same frequency, (2) Equation 1 often predicts reasonably well the value of RRS_{max} (Dobry 1991), and (3) average values of RRS and (a_A / a_B) for the same period range are within 30% of each other (Joyner et al. 1994). Therefore, based on Equation 1 it could be expected that the value of RRS_{max} at soft clay sites is controlled by two main factors: the impedance ratio $I = (\gamma_r / \gamma_s)(V_r / V_s)$, and the internal damping β_s of the soil. Some main reasons why the Mexico City clay exhibits unusually high amplifications are the unusually low values of V_s, γ_s and β_s of this soil, in addition to its extremely linear stress-strain response in shear. For most soft clay sites such as those typically encountered in the United States, the values of β_s and γ_s are higher (Dobry 1991); in fact the ratio $\gamma_r / \gamma_s \approx 1.1$ to 1.4 is quite constant for most sites, while β_s depends mostly on the intensity of the rock motions (due to soil nonlinearity) and on the plasticity index of the clay (Vucetic and Dobry 1991). Thus, for a given type of rock outcrop, I is about proportional to $1/V_s$, and for a relatively narrow range of plasticity indices, it should be expected that RRS_{max} at a site would depend mainly on V_s and on the intensity of the rock motions. This use of Equation 1 also predicts that for a given clay deposit of very high plasticity and low soil nonlinearity (and correspondingly small value of β_s), RRS_{max} should depend mainly on V_s; this has been confirmed by the Mexico City experience (Ordaz and Arciniegas 1992, Dobry et al. 1997).

Seismic code provisions based on the theoretical framework described above, that is, on calculations of the site period and of RRS_{max}, may be appropriate for a specific area consisting mainly of soft clays of known depth on much stiffer soil or rock, and for expected earthquakes of long duration that induce soil resonance without much soil nonlinearity. This is the case for the Mexico City seismic code (Simón and Suarez 1994). However, U.S. codes must consider a much wider variety of site conditions and earthquake motions of various durations and frequency contents, for which a direct application of the model of Figure 4 either is not relevant or is impractical. In many soft clay sites there is often the complication of stiffer soil layers between the soft clay and the rock (Figure 3). Also, in soft clay sites the values of site period and RRS_{max} will typically depend on the earthquake. Therefore, a more practical approach for the evaluation of site coefficients for seismic codes is the calculation of an average (or average plus one standard deviation) value of RRS at a given period for a number of stations having generally similar soil conditions, as done by Joyner et al. (1994), see Figure 2. Based on the assumption that the energy of the wave is preserved, Joyner et al. (1981) have suggested that the site amplification at a given period, as measured by RRS, should be approximately proportional to $(V_s)^{-0.5}$, where V_s is again the shear wave velocity of the soil at shallow depth.

Empirical calculations show that average amplification factors calculated using Ratios of Fourier Spectra between soil sites and nearby rock sites, are proportional to the mean shear wave velocity of the top 30 m, $\overline{V}_s$, raised to a (typically negative) exponent; with this exponent depending on the period band and on the amplitude of the rock ground motion (Borcherdt 1993, 1994a). These results are consistent with those derived from other data sets by Borcherdt and Glassmoyer (1994), Midorikawa et al. (1994) and Boore et al. (1994, 1997), using either RRS or Ratios of Fourier Spectra. For low amplitude rock motions, 0.1g and less, these results by Borcherdt show that the corresponding amplification factors are approximately proportional to $(\overline{V}_s)^{-0.4}$ for short periods, and to $(\overline{V}_s)^{-0.6}$ for periods about 1 second and longer. Corresponding regression curves fitted to the average Ratio of Fourier Spectra computed for a short-period band (0.1 - 0.5 sec) and for a longer-period band (0.4 - 2.0 sec) are shown in Figure 6. Figure 6 indicates values of short- and long-period amplification factors, F_a and F_v, decreasing toward 1.0 as $\overline{V}_s$ approaches a value of about 1000 m/sec or greater corresponding to the reference rock sites.

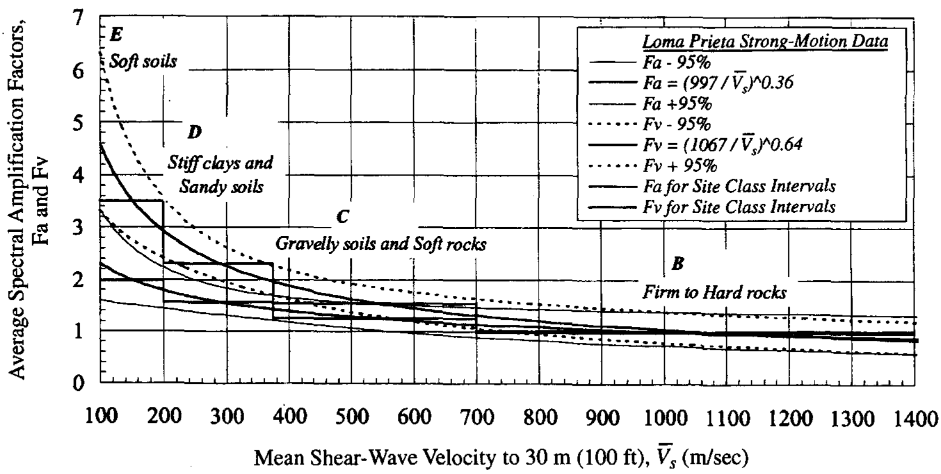

Figure 6. Short-period F_a and mid-period (termed long-period in code provisions) F_v amplification factors with respect to Firm to Hard rock (B) plotted as a continuous function of mean shear-wave velocity, $\overline{V}_S$,using the regression equations derived from the Loma Prieta earthquake from Ratios of Fourier Spectra of nearby soil and rock records. The 95% confidence intervals for the ordinate to the true population regression line and the limits for +2 standard error of estimate are shown. Corresponding amplification factors predicted for the simplified site classes with respect to Firm to Hard rock also are shown (Borcherdt 1994b).

SITE COEFFICIENTS AND SITE CLASSIFICATIONS PRIOR TO 1994

This section considers the basis for the regulations on local site conditions contained in building seismic codes prior to 1994, such as the 1991 version of the *NEHRP Provisions*. The next section discusses the new site categories and site coefficients for seismic design of buildings, incorporated into the 1994 and 1997 *NEHRP Provisions* and the *1997 UBC.*

Seed et al. (1976a), and more recently Idriss (1990, 1991), studied the relation between peak acceleration recorded on soil and that obtained on a nearby rock outcrop. Seed and his coworkers had at their disposal mostly records on stiff and deep cohesionless soil and found little difference between rock and soil accelerations for those sites. Figure 7, on the other hand, includes the more recent curve developed by Idriss for soft sites. For low rock accelerations, the curve is based on records at distant sites obtained during the 1985 Mexico City and 1989 Loma Prieta earthquakes; site response calculations were used by Idriss (1990, 1991) to extrapolate the curve to larger rock accelerations. For low rock accelerations of the order of 0.05g to 0.10g, the corresponding soft soil accelerations are 1.5 to 4 times greater than the rock accelerations (that is, RRS $\approx$ 1.5 to 4 for T = 0). This amplification factor decreases as the rock acceleration increases, and it becomes approximately unity for a rock acceleration of 0.4g, with a tendency for deamplification to occur at larger rock accelerations.

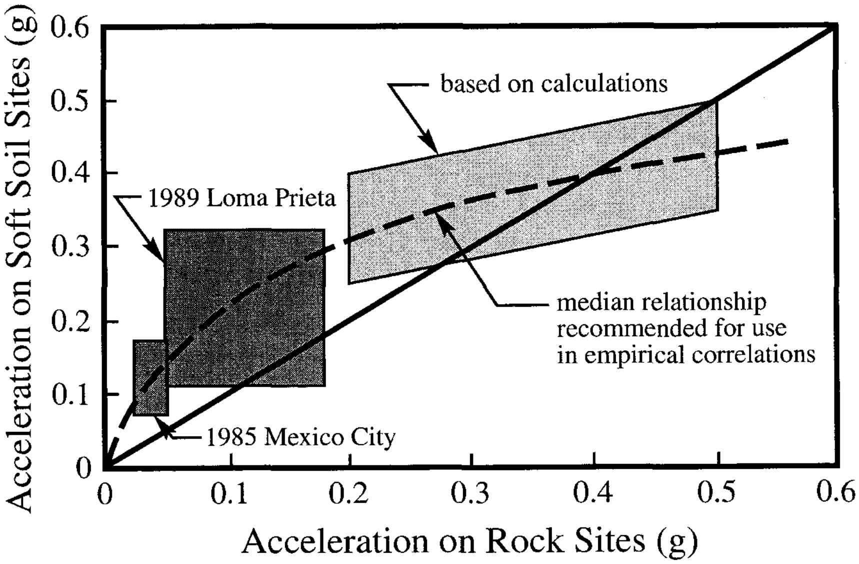

Figure 7. Relationships between maximum acceleration on rock and other local site conditions (Idriss 1990, 1991). This amplification at low rock motion intensities, and lack of amplification or even deamplification at high rock motion amplitudes, is directly related to the nonlinear stress-strain behavior of the soil as the rock acceleration increases.

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A second step in the development of seismic regulations prior to 1994 was the study of the shape of the response spectrum and its correlation with the site conditions. Average spectral shapes for various soil conditions were developed by Seed et al. (1976a, b), on the basis of a statistical study of more than 100 records from twenty-one, mostly California earthquakes available at the time. These spectral shapes are shown in Figure 8. As they are anchored at T = 0 to a value of 1.0, their use requires knowing the peak acceleration (or "effective peak acceleration") on rock or soil, which could be obtained, for example, from seismic hazard maps. The spectral shapes in Figure 8 are fairly constant and independent of site conditions at short periods, but they are very different at longer periods, $T > 0.5$ sec. Therefore, based on these and other similar studies (e.g., Mohraz 1976), the *ATC 3* project (Applied Technology Council, 1978) developed the simplified response spectrum shapes shown in Figure 9. These simplified spectral shapes were later incorporated into the *Uniform Building Code*. It can be noted in Figure 9 that the Ratios of Response Spectra, RRS, relative to rock or shallow stiff soil (i.e., relative to soil profile type S1) were approximately equal to 1.5 for soil type S2 (deep firm soils) and 2.2 for soil type S3 (soft soils 20 to 40 feet thick). The response spectral shapes in Figure 9 are good idealizations of the findings of Seed et al. (1976a,b) shown in Figure 8.

The *ATC 3* project also developed recommendations for lateral force coefficients. In the process of developing these recommendations, the slope of the long-period branch of the spectrum was changed from the decline as 1/T in the spectra (Figure 9) to a flatter slope that declined as $1/T^{2/3}$. The 1/T decline is more typical of the statistics of actual ground motion spectra. The flattening of the long-period branch of the site coefficient curve was accompanied by a reduction in the site factors as well as a multiplication of the curves by a factor of 1.2 (in the *NEHRP Provisions* as well as in the seismic provisions for bridges in AASHTO[2] 1996) and by a factor of 1.25 (in the *UBC*). The resulting site factors are summarized in Table 1 along with the description of the site categories. It can be seen by comparison of Figure 9 and Table 1 that the site factors for soil profile types S2 and S3 were reduced from the response spectral ratios originally developed in *ATC 3* of approximately 1.5 and 2.2 to values of 1.2 and 1.5, respectively. Note also in Table 1 that a soil profile type S4, for deep soft clays, was added to the seismic provisions in the period between 1988 and 1994 (BSSC 1988, 1992), following the large soil amplifications observed in Mexico City during the 1985 earthquake.

As described in the next section, these various factors have been removed from the 1994 and 1997 *NEHRP Provisions* and from the *1997 UBC*. That is, in the new seismic provisions there is no longer a multiplication factor 1.2 nor flattening of the curve, which is now allowed to decline as 1/ T in both rock and soil. As a result, in the new versions of the *NEHRP Provisions* and the *UBC* both the levels and shapes of rock and soil spectra reflect better the actual evidence of ground motions in the free field.

In the *NEHRP Provisions* both before and after 1994, the hazard on rock for any area in the United States is defined by two seismic parameters obtained from separate hazard maps (Algermissen and Perkins 1976; BSSC 1988, 1992, 1995, and 1998; see also Frankel et al. 2000, Holmes 2000, and Leyendecker et al. 2000 in this volume). These two mapped parameters allow construction of the short and long period parts of the spectrum. Prior to 1997, these two parameters were the "effective peak accelerations" A_a and A_v, which ranged in value from less than 0.1g in low hazard areas to 0.4g in high hazard areas in the western United States. The values of A_a and A_v from the maps were used to construct the rock spectrum using a normalized spectral shape (e.g., BSSC 1995), either to define a lateral force

2. American Association of State Highway and Transportation Officials.

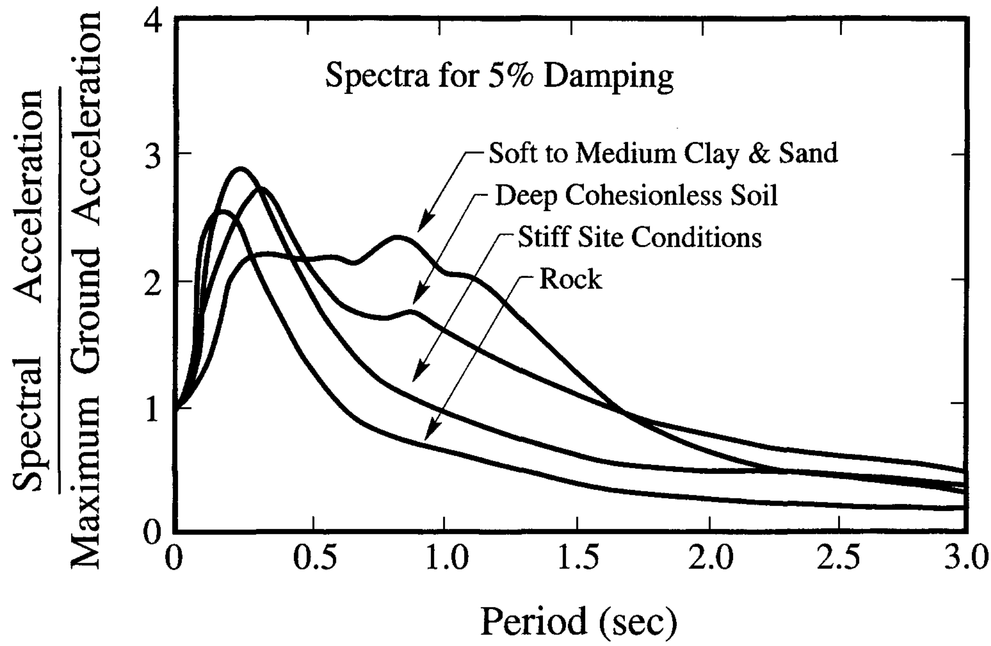

Figure 8. Average acceleration spectra for different site conditions (Seed et al. 1976a,b).

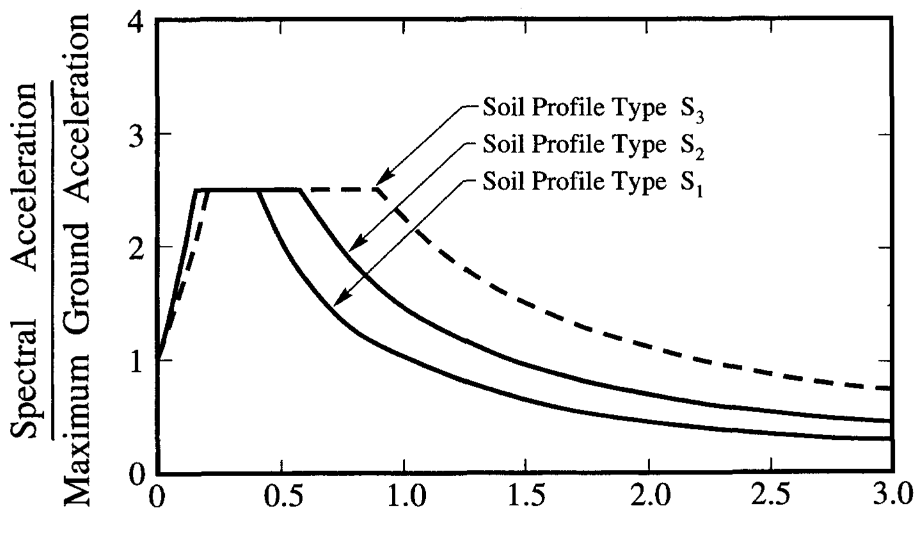

Figure 9. Spectral shapes proposed by *ATC 3* (1978) for soil profile types S1, S2, and S3 (see Table 1).

Table 1. Soil profile types and site factors for calculation of lateral force contained in seismic codes prior to the *1994 NEHRP Provisions* (modified after Martin and Dobry 1994)

Soil Profile Type	Description	Site Coefficient, S
S_1	A soil profile with either (1) rock of any characteristic, either shale-like or crystalline in nature, that has a shear wave velocity greater than 2,500 feet per second or (2) stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying the rock are stable deposits of sands, gravels, or still clays.	1.0
S_2	A soil profile with deep cohesionless or stiff clay conditions where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.	1.2
S_3	A soil profile containing 20 to 40 feet in thickness of soft-to medium-stiff clays with or without intervening layers of cohesionless soils.	1.5
S_4	A soil profile characterized by a shear wave velocity of less than 500 feet per second containing more than 40 feet of soft clays or silts.	2.0

coefficient to compute the lateral force, or to design the structure using modal superposition (see curve labeled "rock" in Figure 10).[3] As discussed in the papers in this volume by Holmes (2000) and Leyendecker et al. (2000), in the *1997 NEHRP Provisions* the A_a and A_v maps have been replaced by two maps of spectral ordinates on rock (Site Class B). A map of the spectral ordinate at T = 0.2 sec, S_s, represents the short period plateau of constant spectral acceleration; while a map of the spectral ordinate at T = 1 sec, S_1, gives the level of the long-period descending curve of spectral acceleration, which is proportional to 1/T. This mapping of spectral ordinates at about 0.2 and 1.0 seconds had been originally recommended by a USGS/SEAOC workshop held in San Francisco in 1988 (Algermissen and Singh 1988).

In principle, the spectral shapes (Figure 9) or long-period site coefficients S (Table 1) used before 1994 could have been applied in conjunction with amplification curves for the peak acceleration such as shown in Figure 7. This would have amplified the short-period spectra, and would have also introduced the effect of soil nonlinearity. However, this was not done, because the studies by Seed et al. (1976a) had found little effect of site conditions on the acceleration. As a result, in the *NEHRP Provisions* before 1994 and the *UBC* before 1997, the soil acceleration was assumed to be equal or close to the rock acceleration. This similarity between soil and rock acceleration seems to be about right on the average for soil sites, including soft clay deposits in highly seismic parts of California, as soil presumably does not amplify or amplifies very little the rock peak acceleration of about 0.4g obtained from the hazard maps (e.g., Figure 7). However, this similarity is certainly not true in the eastern United States and other parts of the country where a level of rock acceleration of 0.1g or 0.2g is obtained from the hazard maps, and where an amplification of the peak acceleration by soft soil of as much as 2 or 3, and corresponding amplification of the short period part of the spectrum, should be considered.

3. In the *UBC*, a single seismic zone factor, Z, is used rather than A_a and A_v (ICBO 1997; Bachman and Bonneville 2000, this volume).

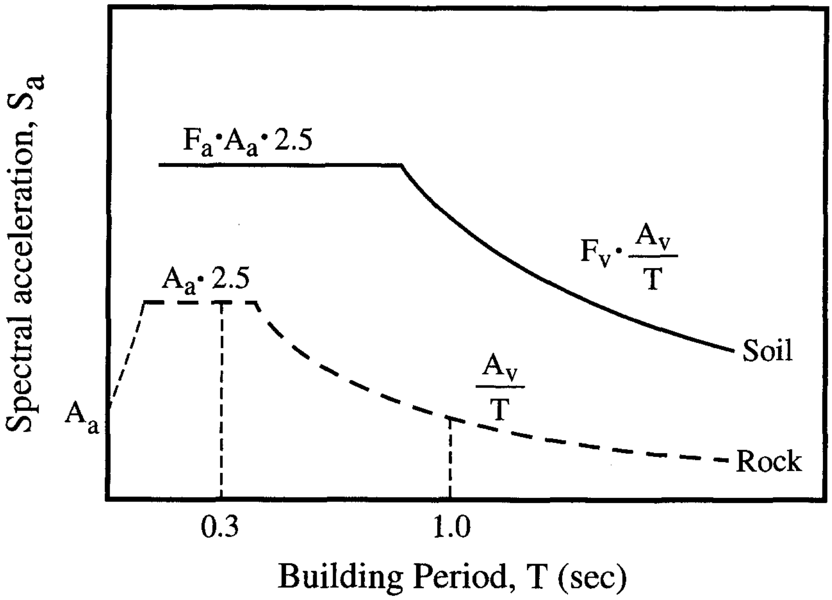

Figure 10. Two-factor approach to local site response incorporated into new seismic codes, starting with the *1994 NEHRP Provisions*.

DEVELOPMENT OF CURRENT SITE FACTORS AND SITE CLASSIFICATIONS

Consensus developed during the site response workshop of November 1992 resulted in the incorporation of a new procedure to account for the effects of site conditions on design spectra in the 1994 version of the *NEHRP Provisions*. This procedure has been incorporated essentially unchanged into the 1997 versions of both the *NEHRP Provisions* and the *UBC*. The new provisions are the result of an effort made over several years, mostly by the geotechnical engineering and earth science communities, and they reflect a broad consensus of both communities based on recorded evidence and theory. This effort is documented in the papers presented to the 1992 workshop, which were included later in the corresponding proceedings (Martin 1994); several of these papers are discussed later in this section. The new procedure and supporting consensus are presented in the 1994 and 1997 *NEHRP Provisions* and the *1997 UBC*, and are discussed in the corresponding commentary volumes (BSSC 1995, 1998). Preliminary reports of the new site categories and site coefficients as well as additional results were published after the workshop by Borcherdt (1993, 1994b), Crouse and McGuire (1996), Martin and Dobry (1994), Rinne (1994), and Seed et al. (1994b).

The new procedure specifies two site coefficients, F_a and F_v, corresponding to the short-period and long-period ranges, respectively, which replace the single long-period site factor S used before. The two site coefficients depend on both site category and intensity of rock motions (defined by A_a or A_v in the *1994 NEHRP Provisions*, by S_s and S_1 in the *1997 NEHRP Provisions*, and by Z in the *1997 UBC*). In addition, each site category (Soil Profile Type in the *1994 NEHRP Provisions* and *1997 UBC*, and site class in the *1997 NEHRP Provisions*) is unambiguously defined by a representative average V_s of the top 30 m of the profile at the site, labeled $\bar{V}_s$ and calculated in a specific way as explained later. Therefore, the old site categories, spectral shapes and site factors of Figure 9 and Table 1 become obsolete and are replaced by the procedure summarized in Figure 10 and Tables 2 and 3. Table 2 includes the approximate correspondence between the new Site Classes A to E and the old site categories S1 to S4. The values of the new site coefficients F_a and F_v are listed in Table 3 and plotted in Figure 11. As summarized in Tables 2 and 3 and listed in more detail later in this article, another site class, F, is also defined for special cases for which F_a and F_v are not specified and which require site-specific evaluations.

Table 2. Summary of site categories in new seismic codes (from 1994 and 1997 *NEHRP Provisions* and *1997 UBC*), including approximate correspondence with old site categories S1 to S4

Site Class or Soil Profile Type	Description	Shear Wave Velocity $\bar{V}_S$ Top 30m (m/sec)	Standard Pen. Resistance $\bar{N}$ or $\bar{N}_{ch}$ (blows/ft)	Undrained Shear Strength $\bar{S}_u$ (kP$_a$)
S1 { A	Hard Rock	> 1500	—	—
S1 { B	Rock	760 - 1500	—	—
S1 and S2 { C	Very dense soil/soft rock	360 - 760	> 50	> 100
S1 and S2 { D	Stiff soil	180 - 360	15 - 50	50 - 100
S3 and S4 { E	Soft soil	< 180	< 15	< 50
S3 and S4 { F	Special soils requiring site-specific evaluation	—	—	—

These values of F_a and F_v as adopted in the recent code provisions (Tables 3a and 3b) and the new definitions of site classes summarized in Table 2, were approved by an extended working group committee at the 1992 workshop. The approved values are based on results derived from both empirical studies of recorded motions and numerical site response analyses. The empirical results included studies of records from the 1989 Loma Prieta earthquake and comparisons with those obtained in previous earthquakes. These data are generally associated with low rock accelerations equal or less than about 0.1g, because results at higher rock motions were not available. The values of site coefficients in Tables 3a and 3b for higher levels of motion are based on laboratory and numerical modeling studies of site response. These empirical, laboratory and numerical studies are summarized in the rest of this section.

Some of the empirical evidence from the 1989 Loma Prieta earthquake and other low rock acceleration events, as processed and interpreted by different authors, has already been discussed in this paper and is illustrated by Figures 1, 2, 6, and 7. The consensus developed at the 1992 workshop started with agreement on this evidence.

Table 3. Site coefficients for short (F_a) and long (F_v) periods as function of site conditions and rock level of shaking contained in new seismic codes (BSSC 1995, 1998)

(a) Short period site coefficient F_a

Site Class Or Soil Profile Type	Mapped Rock Shaking Level at Short Periods				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
	$A_a \leq 0.10$	$A_a = 0.20$	$A_a = 0.30$	$A_a = 0.40$	$A_a \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	*
F	*	*	*	*	*

(b) Long period site coefficient F_v

Site Class or Soil Profile Type	Mapped Rock Shaking Level at Long Periods				
	$S_l \leq 0.10$	$S_l = 0.20$	$S_l = 0.30$	$S_l = 0.40$	$S_l \geq 0.50$
	$A_v \leq 0.10$	$A_v = 0.20$	$A_v = 0.30$	$A_v = 0.40$	$A_v \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	*
F	*	*	*	*	*

(*) Site-specific geotechnical investigation and dynamic site response analyses shall be performed

A key role was played by statistical studies of Ratios of Fourier Spectra and RRS of the Loma Prieta earthquake for several site conditions normalized to nearby rock sites, as reported by Borcherdt and Glassmoyer (1992), Borcherdt (1994a) and Joyner et al. (1994). The empirical correlations between site coefficients F_a and F_v and wave velocity $\bar{V}_s$ of the site for the 1989 Loma Prieta earthquake, already presented in Figure 6 and repeated again in Figure 12 (curves labeled 0.1g), helped define the values of F_a and F_v in Table 3 for a low value of the effective peak accelerations, $A_a = A_v = 0.1$. In both Figures 6 and 12, F_a and F_v are calculated as average Ratios of Fourier Spectra between a soil and nearby firm to hard rock station, with these averages calculated, respectively, in the period bands 0.1-0.5 sec and 0.4-2.0 sec. The regression curves included in Figures 6 and 12 were prepared by Borcherdt (1993; 1994a, b). Additional calculations in these references for other period bands (0.5-1.5 and 1.5-5.0 sec) showed that the way F_a and F_v are defined in Figures 6 and 12 is sufficient to characterize the local site response within a two-factor approach. These estimates were consistent with similar estimates based on RRS for similar period bands computed for alluvial and bay mud sites by Joyner et al. (1994), with some discrepancies observed at the longer-period bands.

These values of F_a and F_v, obtained directly from records, were used to calibrate numerical one-dimensional site response analytical techniques, including equivalent linear as well as nonlinear programs. The equivalent linear program SHAKE (Schnabel et al. 1972), which had been shown in previous studies to provide reasonable predictions of soil

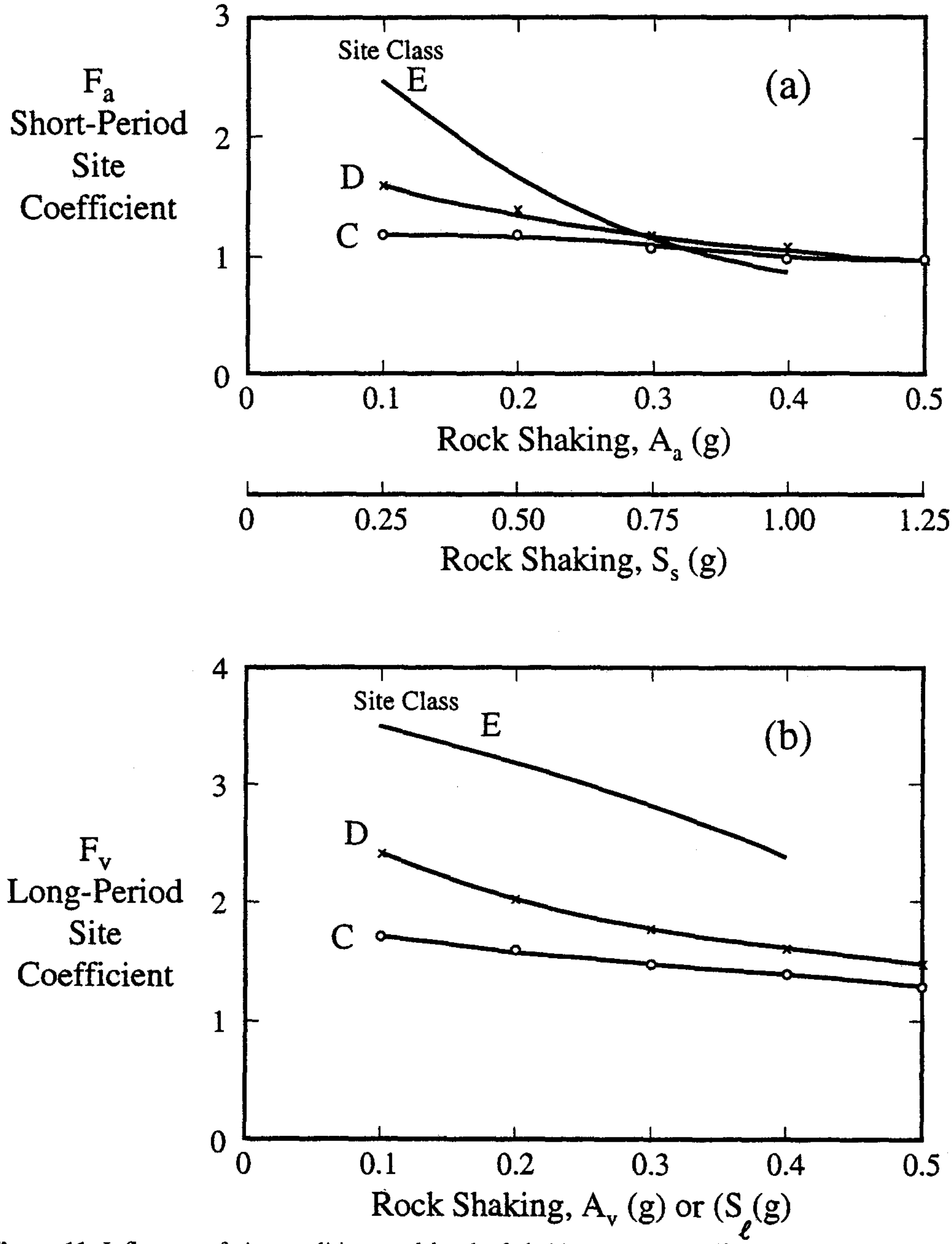

Figure 11. Influence of site conditions and level of shaking on site coefficients F_a and F_v (BSSC 1995, 1998).

amplification during earthquakes (e.g., Seed and Idriss 1982), was used extensively for this calibration. Seed et al. (1994a) and Dobry et al. (1994) showed that the one-dimensional model provided a good first-order approximation to the observed site response in Loma Prieta, especially at soft clay sites.

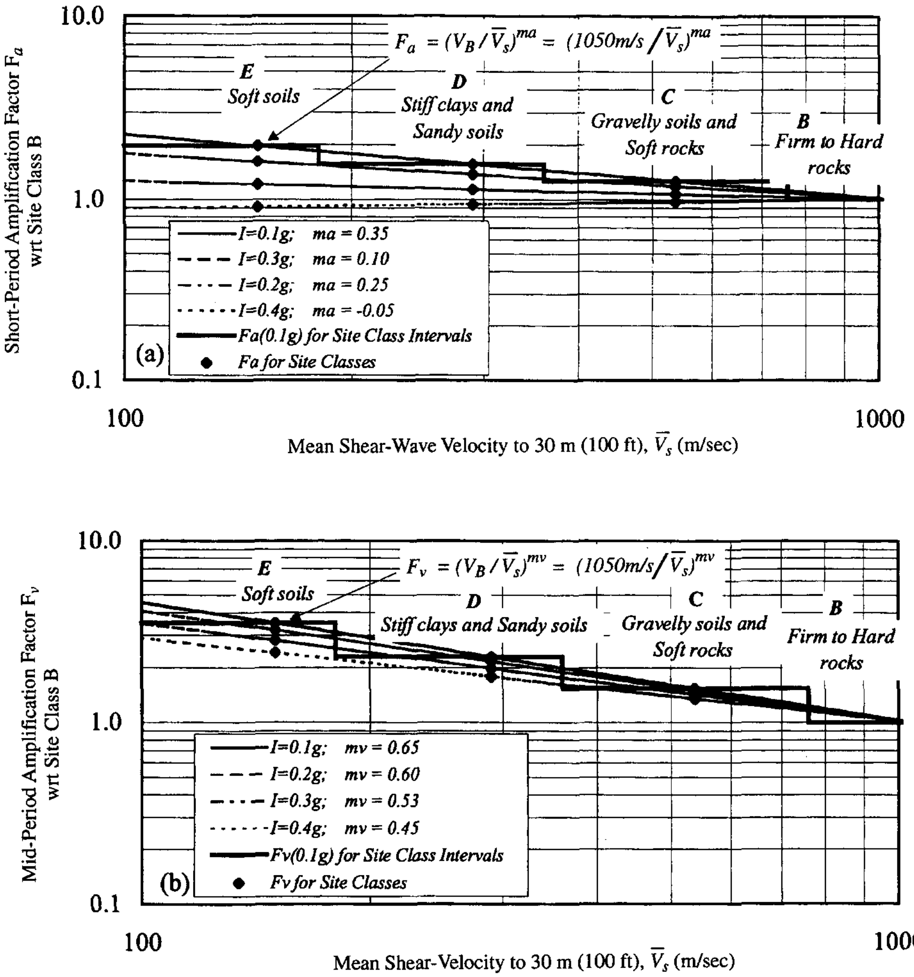

Figure 12. (a) Short-period F_a and (b) mid-period (termed long-period in code provisions) F_v amplification factors with respect to Firm to Hard rock (B) plotted with logarithmic scales as a continuous function of mean shear-wave velocity, $\bar{V}_S$, using the indicated equations with exponents m_a and m_v for the appropriate level of input ground motion (Equations 2 or 4, Borcherdt 1994b). Amplification factors with respect to B for the simplified site classes also are indicated. The plots show that the equations represent straight lines through the points determined by the logarithms of the amplification factors and shear velocities for the Soft-soil (E) and Firm to Hard rock (B) site classes. The exponents m_a and m_v represent the slope of the straight lines and can be modified easily as new information on the amplification characteristics of Soft-soil deposits becomes available. "I" in this figure corresponds to the input ground motion level on rock expressed in units of g (Borcherdt 1994b).

Finally, these calibrated equivalent linear and nonlinear one-dimensional site response techniques were used to extrapolate the values of F_a and F_v to larger rock accelerations of as much as 0.4g or 0.5g, through parametric studies using both equivalent linear and nonlinear analyses. Those parametric studies involved combinations of hundreds of soil profiles and soil properties and several dozen input earthquake rock motions scaled to peak rock accelerations between less than 0.1g and more than 0.4g (Seed et al. 1994a, Dobry et al. 1994).

Figure 13 summarizes some results of these site response analyses using the equivalent linear method. This figure presents values of peak amplification of response spectra at long periods for soft sites, RRS_{max}, calculated using the equivalent linear approach as a function of the plasticity index (PI) of the soil, and of the rock wave velocity V_r, for both weak ($A_a = A_v = 0.1g$) and strong ($A_a = A_v = 0.4g$) input rock shaking. The effect of PI is due to the fact that soils with higher PI exhibit less stress-strain nonlinearity and a lower material damping β_s (Vucetic and Dobry 1991). For $A_a = A_v = 0.1g$, $V_r = 4{,}000$ ft/sec (1220 m/s) and PI = 50, roughly representative of San Francisco Bay area soft sites in the Loma Prieta earthquake, $RRS_{max} = 4.4$, which for a soil shear wave velocity of 150 m/sec coincides with the upper part of the range in Figures 6 and 12 inferred from the records. Note the reduction of this value of RRS_{max} from 4.4 to about 3.3 in Figure 13 when $A_a = A_v = 0.4g$ due to soil nonlinearity.

The form of the regression curves in Figures 6 and 12 for rock accelerations of about 0.1g, together with the numerical site response analyses by Seed et al. (1994a) and Dobry et al. (1994), suggested a simple and well-defined procedure for extrapolation of the Loma Prieta results. Figure 12 shows that the functional relationship between the logarithms of amplification and mean shear velocity is a straight line (Borcherdt 1994a, b). That is, the amplification is controlled by the amplification for two site classes, chosen as "soft soil" and "firm to hard" rock. With the amplification factor for "firm to hard" rock being unity, the extrapolation problem is determined by specification of the amplification factors at successively higher levels of motion for the soft-soil site class, as inferred from the site response analyses. Curves and corresponding equations predicting the amplification factors for higher levels of motion are shown in Figures 12a and 12b. The curves suggest that the average amplification at a site is proportional to the "rock-soil" ratio of shear wave velocities raised to an exponent ma or mv. The exponents are defined by the slope of the straight line determined by the logarithms of the amplification factors and the shear velocities for the soft-soil and the "firm to hard" rock site classes. The exponents vary with level of input rock motion. Using mean estimates of shear wave velocity for the site classes and corresponding equations as specified in Figures 12a and 12b yields estimates of amplification factors F_a and F_v within 0.1 of those approved at the 1992 workshop and included in Table 3.

As explained in more detail in the next section, in a typical profile where V_s varies with depth, $\bar{V}_s$ is obtained from the travel time of a vertically propagating shear wave between a depth of 30 m and the ground surface. The discussion on basic considerations earlier in this paper clearly showed that use of the shear wave velocity of the soil for this purpose is scientifically sound. Correlations with soil amplification in specific earthquakes, such as shown for Loma Prieta in Figure 6 (Borcherdt 1994a) and for the 1985 Mexico City earthquake by Ordaz and Arciniegas (1992) and Dobry et al. (1999), have confirmed the usefulness of $\bar{V}_s$ to predict soil amplification. The specific ranges of $\bar{V}_s$ used to define the site categories in Table site categories in Table 2 were based on analyses of several extensive sets of borehole geotechnical data (Gibbs et al. 1975; Fumal 1978; Fumal and Tinsley 1985; and Borcherdt 1994a, b); these ranges of $\bar{V}_s$ in Table 2 also reflect slight modifications adopted for consistency with previous boundaries.

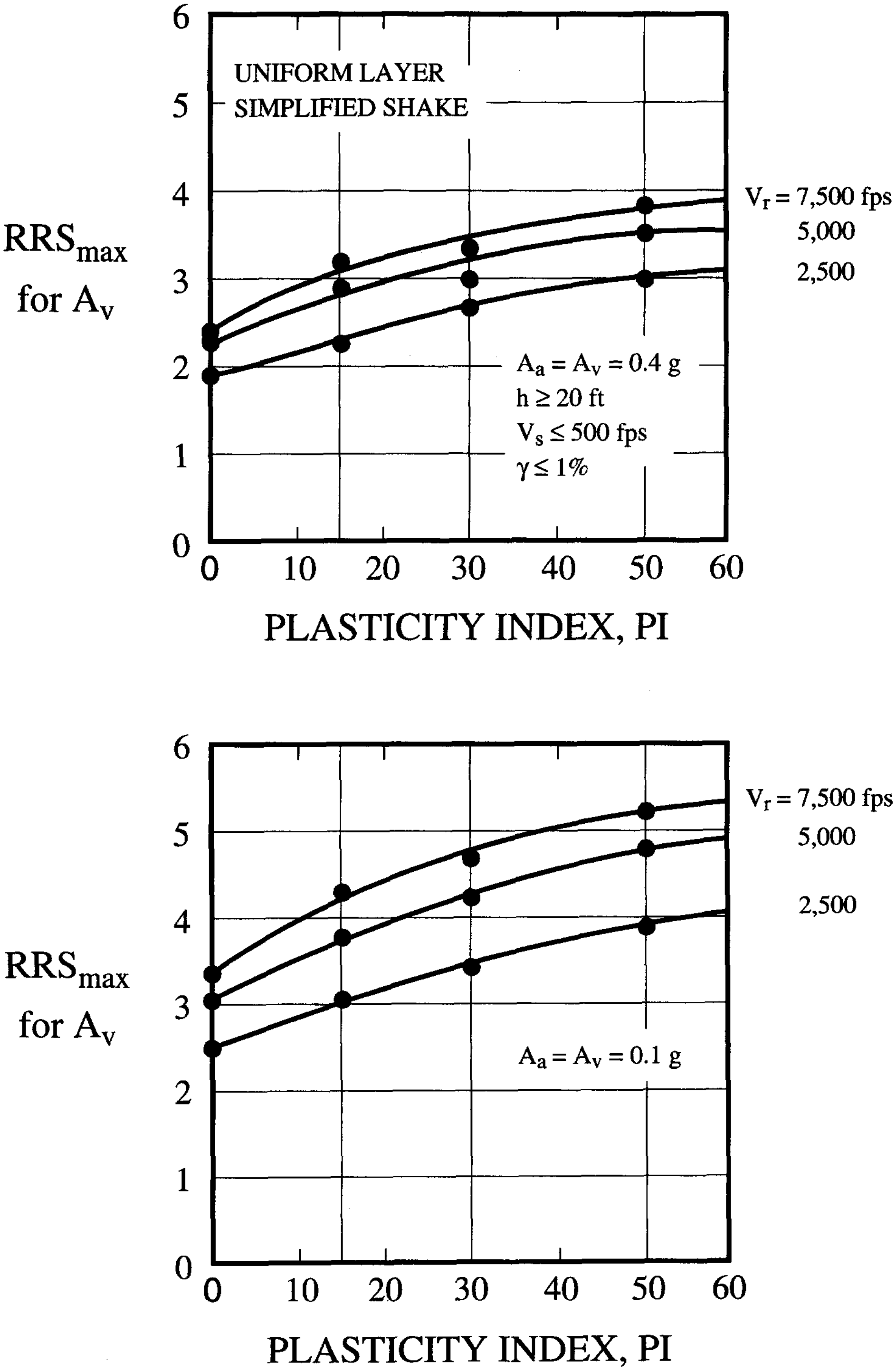

Figure 13. Variation of RRS_{max} of uniform layer of soft clay on rock from equivalent linear site response analyses (Dobry et al. 1994).

DISCUSSION OF PRINCIPAL CHANGES IN SITE COEFFICIENTS AND SITE CATEGORIES

This section reviews again in some detail the main changes contained in the 1994 and 1997 *NEHRP Provisions* and the *1997 UBC*, now mainly from the viewpoint of the user. These changes include the switch from a one- to a two-factor approach for site effects through the introduction of the short-period site coefficient F_a; the dependence of both site factors on the level of rock motions to consider the effect of soil nonlinearity; and the use of $\bar{V}_s$ of the top 30 m to characterize the site. When comparing the 1994 and 1997 *NEHRP Provisions*, the changes in the way the seismic hazard on rock is specified must also be considered.

ACCELERATION SPECTRUM, SITE COEFFICIENTS, AND SEISMIC HAZARD

The sketch of spectral acceleration, S_a, in Figure 10 was prepared for the *1994 NEHRP Provisions* using A_a and A_v as measures of seismic hazard on rock. The plot is still conceptually valid for the *1997 NEHRP Provisions* if A_a is replaced by $S_s = 2.5\ A_a$, giving the height of the short-period segment of constant spectral acceleration on rock. Similarly, A_v in Figure 10 should be replaced by $S_1 = A_v$ giving the rock spectral ordinate at a one-second period, and thus also establishing the position of the long-period descending curve of hyperbolic shape, $S_a = S_1 / T$. That is, in both rock and soil, the spectrum considered by the *1997 NEHRP Provisions* (referred to there as maximum considered earthquake spectral response acceleration) is $S_a = F_a\ S_s$ at short periods and $S_a = F_v\ S_1 / T$ at long periods. The design spectrum in the *1997 NEHRP Provisions* is proportional to these values of S_a (Leyendecker et al. 2000, Holmes et al. 2000, this volume). Therefore, the seismic design of structures at both soil and rock sites is controlled by the products $F_a\ S_s$ and $F_v\ S_1$, with the values of site coefficients F_a and F_v given by Table 3. As the values of F_a and F_v are greater in areas of small or medium seismic hazard corresponding to relatively small S_s and S_1, these products are not so different for comparable soil sites located in areas of low and high seismic hazard. This conclusion is especially true if the comparison is done for soft soil sites; it can be easily verified in Table 3 by comparing two soft sites classified as E, one in a high seismic hazard area ($A_a = 0.4g$ or $S_s = 1g$) and the other in a moderate hazard area ($A_a = 0.2g$ or $S_s = 0.5g$). The products $F_a\ S_s$ for Site Class E are (1) (0.9) = 0.9g and (0.5) (1.7) = 0.85g, that is, they are essentially identical. The dramatic amplification of the level of seismic ground motions by soft soil sites in low or moderate seismicity areas tends to erase the traditional concept of seismic hazard focused exclusively on motions on rock or firm ground. This large amplification of weaker rock motions is of course a direct consequence of the nonlinear response of soil illustrated by Figures 7 and 13, which tends to amplify more the rock motions when they are smaller.

SITE CHARACTERIZATION

The site characterization (site class) is now based only on the top 30 m (100 ft) of soil (Table 2), disregarding the depth of soil to rock if greater than 30 m, the soil properties below 30 m, and the properties of the rock underlying the soil.[4] The site class is determined solely

4. One case in which the soil properties below 30 m are considered is in a soil deposit having very thick soft/medium stiff clays more than 36 m in thickness, which is always classified as Site Class F, requiring site-specific evaluation.

and unambiguously by one parameter (the representative average shear-wave velocity to a depth of 30 m).[5] This representative average V_s to 30 m, denoted in the provisions as $\bar{V}_s$, is not computed as the arithmetic average of values of V_s down to that depth. Rather, $\bar{V}_s$ is calculated from the time taken by the shear wave to travel from a depth of 30 m to the ground surface. Specifically, for a profile consisting of n soil or rock layers each having a shear-wave velocity V_s and thickness h: $\bar{V}_s = 30 / [\Sigma(h/V_s)]$, where the sum is done between ground surface and 30 m. (The purpose of calculating $\bar{V}_s$ this way is to be able to classify a soft soil deposit on rock as Site Class E, even when the depth to rock is less than 30 m.) To illustrate this calculation of $\bar{V}_s$, let us take the soil profile of Figure 3, which contains two layers of soft clay and alluvium above 30 m depth. In this case, $\bar{V}_s = 30 / (18/90 + 12/260) = 122$ m/sec < 180 m/sec, and thus the site is classified in Table 2 as Site Class E. Limiting the depth to 30 m and using $\bar{V}_s$ as specified above define an unambiguous standard for site characterization.

The use of soil properties sometimes more readily available than $\bar{V}_s$, such as the Standard Penetration Resistance ($\bar{N}$ or $\bar{N}_{ch}$ in Table 2), or undrained shear strength ($\bar{s}_u$ in Table 2), is also conservatively allowed in the new provisions to characterize the top 30 m of soil. Previous code versions, while relying on qualitative descriptions of the soil and thus being more ambiguous, did require information on soil type and total soil thickness down to greater depths (200 ft), as shown by Table 1. While these other parameters in addition to $\bar{V}_s$ of the top 30 m certainly play a role in local site response, it was felt that a single-parameter characterization based on $\bar{V}_s$ was appropriate at this stage and should cover most cases of interest. As discussed earlier, theoretical considerations and studies of actual ground motions point out the significance for site amplification of the values of the shear wave velocities in the shallower part of the soil profile; therefore, if one parameter is to be selected, $\bar{V}_s$ or a similar average is a natural one from a scientific viewpoint. $\bar{V}_s$ is also clearly measurable in the field (by geophysical techniques), thus removing the ambiguity of definitions of site categories contained in previous codes. Finally, the restriction to the top 30 m makes it much more feasible for geotechnical engineers and earth scientists to obtain the necessary information for the site from available data. Therefore, the site classes based on $\bar{V}_s$ are now unambiguous, practical to use, and scientifically sound in that site amplification of ground motions for many or most soil sites and earthquakes are expected to be determined in the first approximation by the value of $\bar{V}_s$. Also, this parameter has provided researchers with an unambiguous and measurable quantity for empirical studies of future earthquake data aimed at verifying, refining, or modifying Table 3, for example, by including other factors in addition to $\bar{V}_s$ as more data become available.

In addition to these site classes, A through E, for which values of the site coefficients F_a and F_v are specified in Table 3, another site class, F, is defined for special soils that could not be covered by the new provisions. No values of F_a and F_v are provided for soils in Site Class F, which require site-specific evaluations. These special cases in Site Class F include (1) soils

5. There are some exceptions to this general rule. For example, a profile with more than 3 m of soft clay is classified as Site Class E, regardless of the average $\bar{V}_s$ of the top 30 m.

vulnerable to potential failure or collapse under seismic loading, such as liquefied soils, quick and highly sensitive clays, and collapsible weakly cemented soils; (2) profiles including a total thickness of 3 m (10 ft) or more of peat and/or highly organic clay; (3) profiles including a total thickness of 8 m (25 ft) or more of very high plasticity clays of plasticity index, PI > 75; and (4) profiles including a total thickness of 36 m (120 ft) or more of soft/medium stiff clays. Cases (3) and (4) were included in Site Class F so as to exclude from Table 3 soil profiles approaching the Mexico City case, where amplifications much greater than those listed in Table 3 are possible.

SHORT-PERIOD SITE COEFFICIENT

A short-period amplification site coefficient (F_a) is introduced, which did not exist before (Table 3a, Figures 10 and 11a). That is, the one-parameter model of local site amplification characterized by the coefficient S is replaced by a two-parameter model characterized by F_a and F_v. Therefore, once the response spectrum on rock is specified (through the values of A_a and A_v in the *1994 NEHRP Provisions*, Z in the *1997 UBC*, or S_s and S_1 in the *1997 NEHRP Provisions*, obtained from the corresponding seismic hazard maps), the spectrum on soil is now calculated by using both F_a (which amplifies the short-period part of the rock spectrum, in the neighborhood of T = 0.2 sec) and F_v (which amplifies the long-period part of the rock spectrum, at periods in the neighborhood of T = 1 sec and above). In the old codes and provisions, in effect $F_a \approx 1$, with no soil amplification at short periods. Both F_a and F_v are unity for rock (Site Class B) and become greater as the soil becomes softer as measured by $\bar{V}_s$ and hence the site class evolves through C, D, and E. For the softer sites (Site Class E with $\bar{V}_s$ < 180 m/s), maximum values of $F_a = 2.5$ and $F_v = 3.5$ are specified in Table 3. In all cases, $F_a \leq F_v$ in Table 3, reflecting the generalized experience about soil amplification that brought about the concept of spectral shapes and normalized response spectra contained in codes prior to the *1994 NEHRP Provisions* (Figures 8 and 9). While soil nonlinearity reduces the value of F_a to about 1 or even less than 1 in soft soils subjected to very intense rock motion, such as characterized by $S_s \geq 1$g or $A_a \geq 0.4$g, large amplifications of short-period motions have been observed on soft soils when the rock motions are less intense, e.g., the 1989 Loma Prieta earthquake. The need for $F_a > 1$ at short periods for less intense rock motions is also predicted by site response analyses.

DEPENDENCE OF SITE COEFFICIENTS ON LEVEL OF SEISMIC HAZARD

Consistent with the analytical studies and field evidence, the effect of soil nonlinearity is introduced by making both site coefficients F_a and F_v functions of the level of intensity of rock motions. That is, the two site coefficients F_a and F_v in Table 3 and Figure 11 are now a function of (a) the site class, and (b) the level of rock motion given by A_a or A_v in the *1994 NEHRP Provisions*, S_s and S_1 in the *1997 NEHRP Provisions*, and Z in the *1997 UBC*. This revision should be contrasted with the old codes and provisions, where the site coefficient S depended only on the site category and was unaffected by A_a or A_v. The main consequence of this change is the appearance of some large amplifications at both short and long periods on soft soil, for those parts of the country where the mapped rock ground motions specified by the codes are low, such as in the eastern United States (e.g., F_a = 2.5 and F_v = 3.5 for Site Class E and A_a or $A_v \leq 0.1$g in Table 3). Therefore, in areas of the country where the seismic hazard is low or medium, the effects of site conditions on ground motions are significantly increased when compared to previous codes. This effect of the change in the 1994 and 1997

NEHRP Provisions and in the *1997 UBC* is exactly as intended based on the evidence. On the other hand, for high seismic hazard areas in California where $A_a = A_v = 0.4g$ (or $S_s = S_1 = 1g$), $F_a \approx 1$, as before, more or less independent of the structure being on rock or soil, and with the resulting shapes of the spectra being generally similar to the old spectral shapes of Figure 9.

RECENT AND ONGOING STUDIES

The new site coefficients for structures incorporated into the 1994 and 1997 *NEHRP Provisions* and the *1997 UBC*, involved a number of empirical and analytical studies, as discussed in a previous section of this paper. These studies summarized the situation prior to 1992, including evaluation of available strong-motion records, and much of this material is included in the proceedings of the workshop held at the University of Southern California in November 1992 (Martin 1994).

Both the adoption of the new site categories and site coefficients, and the fast growth in the number of records in the last few years, especially from the 1994 Northridge, California, and 1995 Kobe, Japan, earthquakes, have stimulated the study of the subject after 1992, with much of this work still ongoing. While the Northridge epicentral area included few if any soft sites, it did have many stations on stiffer soils, and the Kobe earthquake had stations on both stiff and soft soils; both earthquakes produced for the first time a wealth of recordings on various site conditions very close to the source of a destructive event. Many studies of these records and of the exact site and topographic conditions at and near the recording stations are being conducted. Much of the effort is oriented to the verification of the new *NEHRP Provisions/UBC* site coefficients listed in Table 3, including the predicted effect of level of rock shaking (soil nonlinearity) at both stiff and soft soil sites, as well as study of other factors not included in the new *NEHRP Provisions* and *UBC*. These other factors, which require further study, and which perhaps should be incorporated in future code developments once their influence is better understood, include influence of the soil and rock below a depth of 30 m, amplification of long-period spectra by deep stiff sites on hard rock, further study of the amplification of nearby earthquakes at shallow stiff sites on hard rock, influence of the shape of the sedimentary valley including basin edge effects and other 2-D/3-D factors, and effect on the site coefficients of near-field ground motions containing only one or a few pulses.

Some of the authors of this paper have been or are currently involved in these post-Northridge and post-Kobe studies (e.g., Borcherdt 1996a,b, 1997; Dickenson and Seed 1997; and Dobry et al. 1997, 1999). It is useful to list here three key references that describe some of the most recent studies conducted by many researchers and summarize preliminary findings. These references are the proceedings volumes of three recent technical meetings: (1) *Proceedings of the International Workshop on Site Response* held in Yokosuka, Japan, in January 1996 (Iai 1996); (2) *Proceedings of the North America–Japan Workshop on the Geotechnical Aspects of the Kobe, Loma Prieta and Northridge Earthquakes* held in Osaka, Japan, in January, 1996 (Bardet et al. 1997); and (3) *Proceedings of the NEHRP Conference and Workshop on Research on the Northridge Earthquake* held in Los Angeles in August 1997 (Mahin 1998).

CONCLUSIONS

1. The new provisions on site effects for structures incorporated into the 1994 and 1997 *NEHRP Provisions* and the *1997 UBC* reflect a broad consensus of the geotechnical engineering and earth science communities and constitute a significant advance over the provisions contained in code versions prior to 1994.
2. The site categories are now based unambiguously on the representative average shear-wave velocity ($\bar{V}_s$) of the top 30 m (100 ft) of the profile at the site. For a site with varying shear-wave velocity between ground surface and 30 m, $\bar{V}_s$ is calculated from the total time needed for the shear wave to travel those 30 m.
3. The main changes in the site coefficients include replacement of the old coefficient S by F_v at long periods; introduction of a new coefficient, F_a, at short periods; and dependence of F_a and F_v on both site category and level of rock shaking to consider soil nonlinearity.
4. Low seismic-hazard areas of the United States are affected more by these changes than high-hazard areas.
5. Ongoing studies of the 1994 Northridge and 1995 Kobe earthquake records at rock and soil sites, as well as future analysis of other earthquake records such as those from the 1999 Turkey and Taiwan earthquakes, are important for additional validation and possible further development or refinement of these new code provisions.

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