Chapter 4 Commentary

GROUND MOTION

4.1 PROCEDURES FOR DETERMINING MAXIMUM CONSIDERED EARTHQUAKE AND DESIGN EARTHQUAKE GROUND MOTION ACCELERATIONS AND RESPONSE SPECTRA: This section sets alternative procedures for determining ground shaking parameters for use in the design process. The design requirements generally use response spectra to represent ground motions in the design process. For the purposes of the *Provisions*, these spectra are permitted to be determined using either a generalized procedure in which mapped seismic response acceleration parameters are referred to or by site-specific procedures. The generalized procedure in which mapped values are used is described in Sec. 4.1.2. The site-specific procedure is described in Sec. 4.1.3.

4.1.1 Maximum Considered Earthquake Ground Motions: The *Provisions* are intended to provide uniform levels of performance for structures, depending on their occupancy and use and the risk to society inherent in their failure. Sec. 1.3 of the *Provisions* establishes a series of Seismic Use Groups that are used to categorize structures based on the specific Seismic Design Category. It is the intent of the *Provisions* that a uniform margin of failure to meet the seismic design criteria be provided for all structures within a given Seismic Use Group.

In past editions of the *Provisions,* seismic hazards around the nation were defined at a uniform 10 percent probability of exceedance in 50 years and the design requirements were based on assigning a structure to a Seismic Hazard Exposure Group and a Seismic Performance Category. While this approach provided for a uniform likelihood throughout the nation that the design ground motion would not be exceeded, it did not provide for a uniform margin of failure for structures designed for that ground motion. The reason for this is that the rate of change of earthquake ground motion versus likelihood is not constant in different regions of the United States.

The approach adopted in the *Provisions* is intended to provide for a uniform margin against collapse at the design ground motion. In order to accomplish this, ground motion hazards are defined in terms of maximum considered earthquake ground motions. The maximum considered earthquake ground motions are based on a set of rules that depend on the seismicity of an individual region. The design ground motions are based on a lower bound estimate of the margin against collapse inherent in structures designed to the *Provisions*. This lower bound was judged, based on experience, to be about a factor of 1.5 in ground motion. Consequently, the design earthquake ground motion was selected at a ground shaking level that is 1/1.5 (2/3) of the maximum considered earthquake ground motion.

For most regions of the nation, the maximum considered earthquake ground motion is defined with a uniform likelihood of exceedance of 2 percent in 50 years (return period of about 2500 years). While stronger shaking than this could occur, it was judged that it would be economically impractical to design for such very rare ground motions and the selection of the 2 percent in

50 years likelihood as the maximum considered earthquake ground motion would result in acceptable levels of seismic safety for the nation.

In regions of high seismicity, such as coastal California, the seismic hazard is typically controlled by large-magnitude events occurring on a limited number of well defined fault systems. Ground shaking calculated at a 2 percent in 50 years likelihood would be much larger than that which would be expected based on the characteristic magnitudes of earthquakes on these known active faults. This is because these major active faults can produce characteristic earthquakes every few hundred years. For these regions, it is considered more appropriate to directly determine maximum considered earthquake ground motions based on the characteristic earthquakes of these defined faults. In order to provide for an appropriate level of conservatism in the design process, when this approach to calculation of the maximum considered earthquake ground motion is used, the median estimate of ground motion resulting for the characteristic event is multiplied by 1.5.

Sec. 4.1.1 of the *Provisions* defines the maximum considered earthquake ground motion in terms of the mapped values of the spectral response acceleration at short periods, S_S, and at 1 second, S_1, for Site Class B sites. These values may be obtained directly from Maps 1 through 24, respectively. A detailed explanation for the development of Maps 1 through 24 appears as Appendix A of this *Commentary* volume. The logic by which these maps were created, as described above and in Appendix A, is also included in the *Provisions* under Sec 4.1.3, Site-Specific Procedures, so that registered design professionals performing such a study may use methods consistent with those that served as the basis for developing the maps.

4.1.2 General Procedure for Determining Maximum Considered Earthquake Ground Motions and Design Spectral Response Accelerations: This section provides the procedure for obtaining design site spectral response accelerations using the maps provided with the *Provisions*. Most buildings and structures will be designed using the equivalent lateral force technique of Sec. 5.4, and this general procedure to determine the design spectral response acceleration parameters, S_{DS} and S_{D1}, that are directly used in that procedure. Some structures will be designed using the modal analysis procedures of Sec. 5.5. This section also provides for the development of a general response spectrum, which may be used directly in the modal analysis procedure, from the design spectral response acceleration parameters, S_{DS} and S_{D1}.

Maps 1 and 2 respectively provide two parameters S_S and S_1, based on a national seismic hazard study conducted by the U.S. Geological Survey. For most buildings and sites, they provide a suitably accurate estimate of the maximum considered earthquake ground shaking for design purposes. For some sites, with special soil conditions or for some buildings with special design requirements, it may be more appropriate to determine a site specific estimate of the maximum considered earthquake ground shaking response accelerations. Sec. 4.1.3 provides guidance on site-specific procedures.

S_S is the mapped value, from Map 1 of the 5 percent damped maximum considered earthquake spectral response acceleration, for short period structures founded on Class B, firm rock, sites. The short period acceleration has been determined at a period 0.2 seconds. This is because it was concluded that 0.2 seconds was reasonably representative of the shortest effective period of buildings and structures that are designed by the *Provisions*, considering the effects of soil compliance, foundation rocking and other factors typically neglected in structural analysis.

Similarly, S_1 is the mapped value from Map 2 of the 5 percent damped maximum considered earthquake spectral response acceleration at a period of 1 second on Site Class B. The spectral response acceleration at periods other than 1 second can typically be derived from the acceleration at 1 second. Consequently, these two response acceleration parameters, S_S and S_1, are sufficient to define an entire response spectrum for the period range of importance for most buildings and structures, for maximum considered earthquake ground shaking on Class B sites.

In order to obtain acceleration response parameters that are appropriate for sites with other characteristics, it is necessary to modify the S_S and S_1 values, as indicated in Sec.4.1.2.4. This modification is performed with the use of two coefficients, F_a and F_v which respectively scale the S_S and S_1 values determined for firm rock sites to appropriate values for other site conditions. The maximum considered earthquake spectral response accelerations adjusted for Site Class effects are designated respectively, S_{MS} and S_{M1}, for short period and 1 second period response. As described above, structural design in the *Provisions* is performed for earthquake demands that are 2/3 of the maximum considered earthquake response spectra. Two additional parameters, S_{DS} and S_{D1} are used to define the acceleration response spectrum for this design level event. These are taken, respectively as 2/3 of the maximum considered earthquake values S_{MS} and S_{M1}, and completely define a design response spectrum for sites of any characteristics.

Sec. 4.1.2.1 provides a categorization of the various classes of site conditions, as they affect the design response acceleration parameters. Sec. 4.1.2.2 describes the method by which sites can be classified according as belonging to one of these Site Classes. Sec. 4.1.2.3 provides definitions of some site parameters referenced in the preceding section.

4.1.2.1 Site Class Definitions: It has long been recognized that the effects of local soil conditions on ground motion characteristics should be considered in building design, and most countries considering these effects have developed different design criteria for several different soil conditions. The 1989 Loma Prieta earthquake provided abundant strong motion data that was used extensively together with other information in developing the 1994 *Provisions*. Evidence of the effects of local soil conditions has been observed globally including eastern North America. An example of the latter is a pocket of high intensity reported on soft soils in Shawinigan, Quebec, approximately 155 miles (250 km) from the 1925 Charlevoix magnitude 7 earthquake (Milne and Davenport, 1969).

The Applied Technology Council (ATC) study that generated the preliminary version of the *Provisions* provided for the use of three Soil Profile Types considered, in the late 1970s, to be different enough in seismic response to warrant separate site coefficients (S factors) and experience from the September 1985 Mexico City earthquake prompted the addition of a fourth Soil Profile Type. These have been revised for the 1994 *Provisions* to conform to the experiences of the Mexico City and the 1989 Loma Prieta earthquake in California as well as to other observations and studies showing the effects of level of shaking, rock stiffness, and soil type, stiffness and depth on the amplification of ground motions at short and long periods. The resulting use of higher seismic coefficients in areas of lower shaking and the addition of a "hard rock" category in the 1994 *Provisions* better reflect the conditions in some parts of the country and incorporate recent efforts toward a seismic code for New York City (Jacob, 1990 and 1991). The need for improvement in codifying site effects was discussed at a 1991 National Center for Earthquake Engineering Research (NCEER) workshop devoted to the subject (Whitman, 1992), which made several general recommendations. At the urging of Robert V. Whitman, a committee was formed during that workshop to pursue resolution of pending issues and develop specific code recommendations. Serving on this committee were M. S. Power (chairman), R. D. Borcherdt, C. B. Crouse, R. Dobry, I. M. Idriss, W. B. Joyner, G. R. Martin, E. E. Rinne, and R. B. Seed. The committee collected information, guided related research, discussed the issues, and organized a November 1992 Site Response Workshop in Los Angeles (Martin, 1994). This workshop discussed the results of a number of empirical and analytical studies and approved consensus recommendations that form the basis for the 1994 *Provisions*.

Amplification of Peak Ground Acceleration: Seed and coworkers (1976a) conducted a statistical study of peak accelerations developed at locations with different site conditions using 147 records from each western U.S. earthquake of about magnitude 6.5. Based on these results, judgment and analysis, they proposed the acceleration relations of Figure C4.1.2-1a that are applicable to any earthquake magnitude of engineering interest. It must be noted that the data base of that study did not include any soft clay sites and, thus, the corresponding curve in the figure was based on the authors' experience and, consequently, was somewhat more speculative.

Idriss (1990a and 1990b), using data from the 1985 Mexico City and 1989 Loma Prieta earthquakes, recently modified the curve for soft soil sites as shown in Figure C4.1.2-1b. In these earthquakes, low maximum rock accelerations of 0.05g to 0.10g were amplified by factors of from about 1.5 to 4 at sites containing soft clay layers ranging in thickness from a few feet to more than a hundred feet and having depths of rock up to several hundred feet. As shown by the data and site response calculations included in Figure C4.1.2-1b, the average amplification factor for soft soil sites tends to decrease as the rock acceleration increases--from 2.5 to 3 at low accelerations to about 1.0 for a rock acceleration of 0.4g. Since this effect is directly related to the nonlinear stress-strain behavior in the soil as the acceleration increases, the curve in Figure C4.1.2-1b can be applied in first approximation to any earthquake magnitude of engineering interest.

It is clear from Figure C4.1.2-1b that low peak accelerations can be amplified several times at soil sites, especially those containing soft layers and where the rock is not very deep. On the other hand, larger peak accelerations can be amplified to a lesser degree and can even be slightly deamplified at very high rock accelerations. In addition to peak rock acceleration, a number of factors including soil softness and layering play a role in the degree of amplification. One important factor is the impedance contrast between soil and underlying rock.

Spectral Shapes: Spectral shapes representative of the different soil conditions discussed above were selected on the basis of a statistical study of the spectral shapes developed on such soils close to the seismic source zone in past earthquakes (Seed et al., 1976a and 1976b; Hayashi et al., 1971).

The mean spectral shapes determined directly from the study by Seed and coworkers (1976b), based on 104 records from 21 earthquakes in the western part of the United States, Japan and Turkey, are shown in Figure C4.1.2-2. The ranges of magnitudes and peak accelerations covered by this data base are 5.0 to 7.8 and 0.04g to 0.43g, respectively. All spectra used to generate the mean curve for soft to medium clay and sand in Figure C4.1.2-2 correspond to rather low peak accelerations in the soil (less than 0.10g). The spectral shapes in the figure also were compared with the studies of spectral shapes conducted by Newmark et al. (1973), Blume et al. (1973), and Mohraz (1976) and with studies for use in model building regulations. It was considered appropriate to simplify the form of the curves to a family of three by combining the spectra for rock and stiff soil conditions leading to the normalized spectral curves shown in Figure C4.1.2-3. The curves in this figure therefore apply to the three soil conditions in the original version (1985) of the *Provisions*.

The three conditions corresponding to the three lines in Figure C4.1.3-3 plus a fourth condition introduced following the 1985 Mexico City earthquake are described as follows:

1. Soil Profile Type 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 ft/s (762 m/s) or (2) stiff soil conditions where the soil depth is less than 200 ft (61 m) and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays.

2. Soil Profile Type S_2--A soil profile with deep cohesionless or stiff clay conditions where the soil depth exceeds 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

3. Soil Profile Type S_3--A soil profile containing 20 to 40 ft (6 to 12 m) in thickness of soft- to medium-stiff clays with or without intervening layers of cohesionless soils.

4. Soil Profile Type S_4--A soil profile characterized by a shear wave velocity of less than 500 ft/sec (152 m/s) containing more than 40 ft (12 m) of soft clays or silts.

The post-Loma Prieta studies (Martin, 1994) have resulted in considerable modification of these profile types resulting in the Soil Profile Types in the 1994 *Provisions*, A through F.

Response of Soft Sites to Low Rock Accelerations: Earthquake records on soft to medium clay sites subjected to low acceleration levels indicate that the soil/rock amplification factors for long-period spectral accelerations can be significantly larger than those in Figures C4.1.2-1 and C4.1.2-2 (Seed et al., 1974). Furthermore, the largest amplification often occurs at the natural period of the soil deposit. In Mexico City in 1985, the maximum rock acceleration was amplified four times by a soft clay deposit that would have been classified as S_4 whereas the spectral amplitudes were about 15 to 20 times larger than on rock at a period near 2 sec. In other parts of the valley where the clay is thicker, the spectral amplitudes at periods ranging between 3 and 4 sec also were amplified about 15 times, but the damage was less due to the low rock motion intensity at these very long periods (Seed et al., 1988). Inspection of the records obtained at some soft clay sites during the 1989 Loma Prieta earthquake indicates a maximum amplification of long-period spectral amplitudes of the order of three to six times.

Figure C4.1.2-4 shows a comparison of average response spectra measured on rock and soft soil sites in San Francisco and Oakland during this magnitude 7.1 earthquake. A preliminary study of the Loma Prieta records at one 285-ft (87 m) soil deposit on rock containing a 55-ft (17 m) soft to medium stiff clay layer (Treasure Island) seems to suggest that the largest soil/rock amplification of response spectra occurred at the natural period of the soil deposit, similarly to Mexico City (Seed et al., 1990).

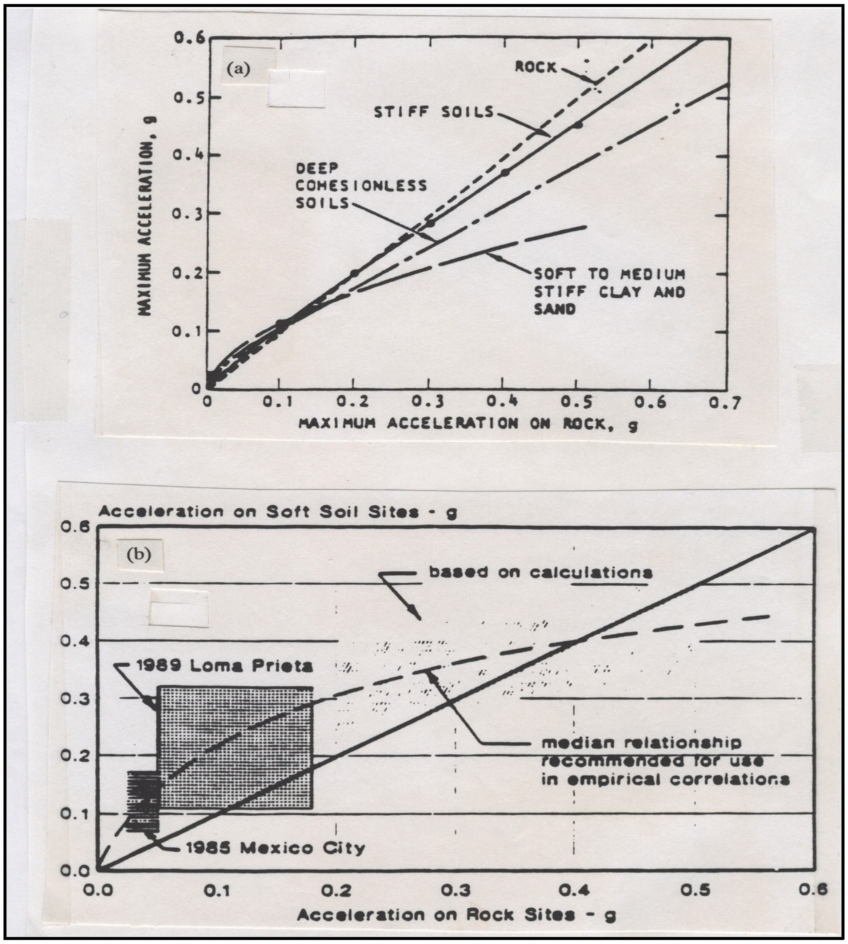


FIGURE C4.1.2-1 relationships between maximum acceleration on rock and other local site conditions: (top) Seed et al., 1976a, and (bottom) Idriss, 1990a and 199b.

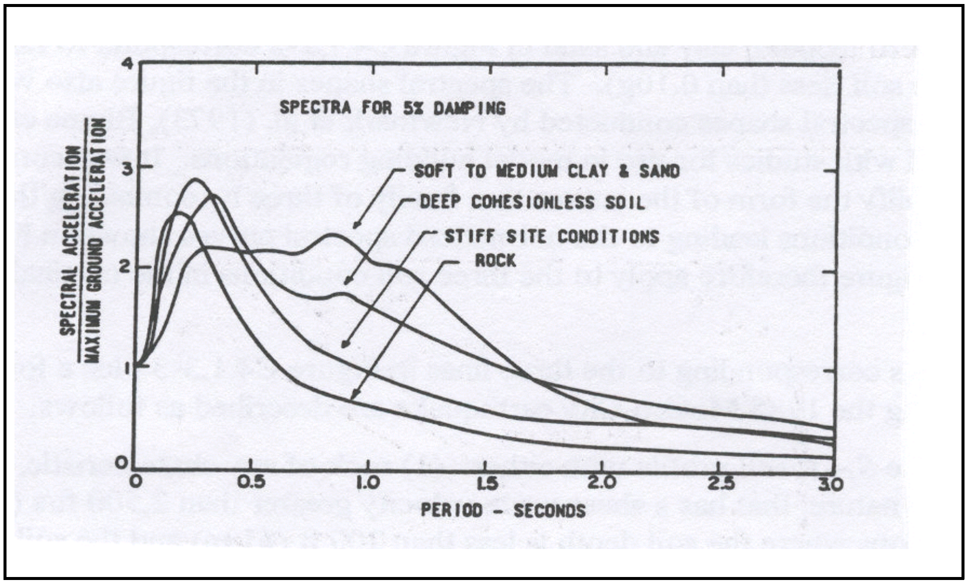


FIGURE C4.1.2-2 Average acceleration spectra for different site conditions (Seed et al., 1976a and 1976b).

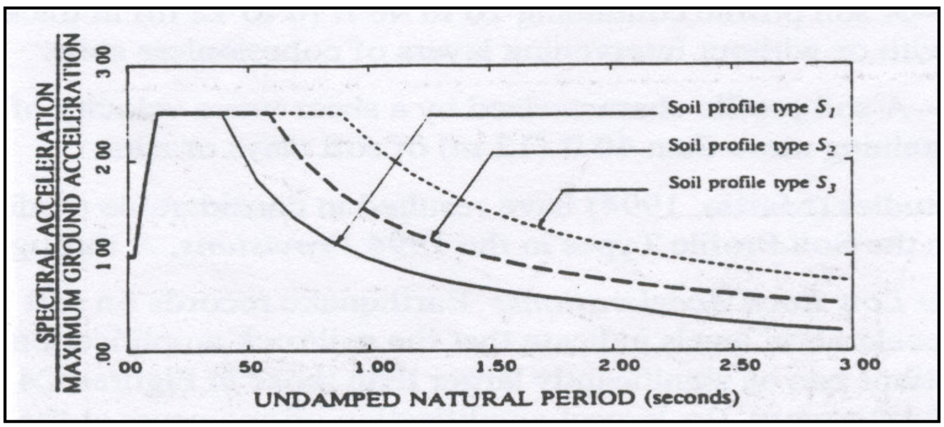


FIGURE C4.1.2-3 Normalized response spectra, damping = 0.05.

Some relevant theoretical and experimental findings are reviewed briefly below to clarify the role of key site parameters in determining the magnitude of the soil/rock amplification of spectral ordinates at long periods for sites containing soft layers. These parameters are the thickness of the soft soil, the shear wave velocity of the soft soil, the soil/rock impedance ratio (*IR*), the layering and properties of the stiffer soil between soft layer and rock, and the modulus and damping properties of the soft soil. The basic assumptions used are those typically used in one-dimensional site response analyses and, thus, the conclusions drawn are restricted to sites where these conditions are fulfilled (i.e., flat sites with horizontal layering of significant extension and far from rock outcrops and with a clear soil-rock inter- face at a depth not exceed- ing several hundre d feet).

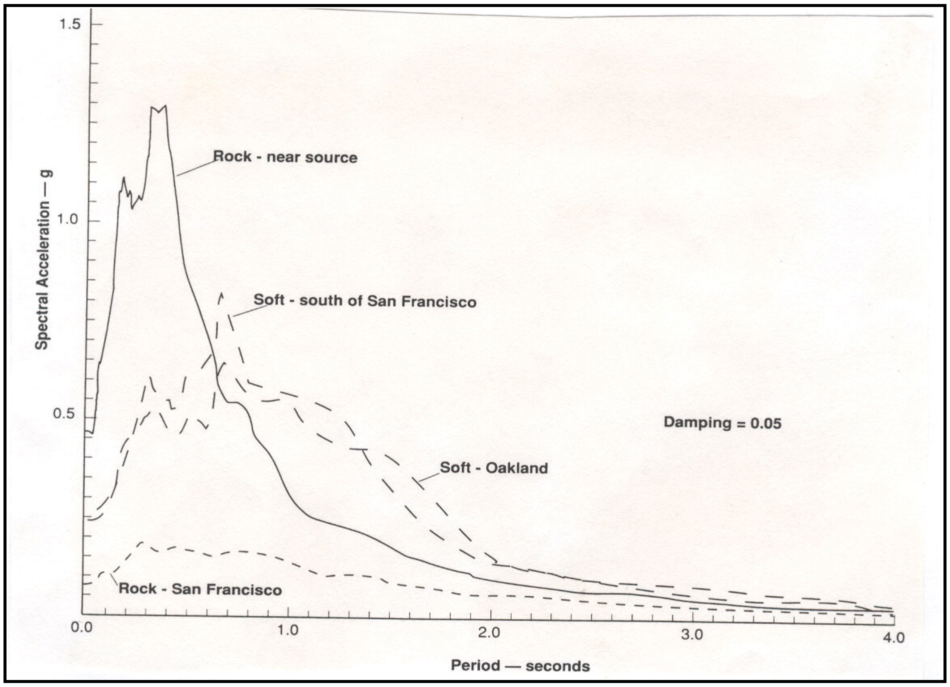


FIGURE C4.1.2-4 Average spectra recorded during 1989 Loma Prieta earthquake at rock sites and soft soil sites (Housner, 1990).

The uniform layer on elastic rock sketched in Figure C4.1.2-5 is subjected to a vertically propagating shear wave representing the earthquake. The soil layer is assumed to behave linearly and it has a thickness h, total (saturated) unit weight g_s, shear wave velocity v_s, and internal damping ratio b_s. The rock has total unit weight g_r, shear wave velocity v_r, and zero damping.

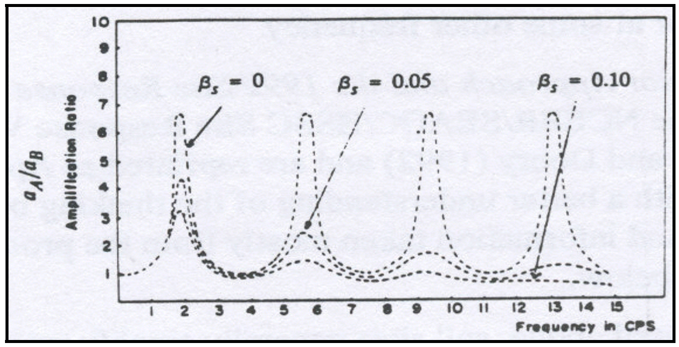


FIGURE C4.1.2-6 Amplification ratio soil/rock for *h* = 100 ft (30.5m), V_s =1.88 cps, and *IR* =6.7 (Roesset, 1977).

Due to the soil-rock interaction effect, the motion at the soil-rock interface C is different (typically less) from that at the rock outcrop B. Only if the rock is rigid (v_s = ¥) are the motions at C and B equal. Of interest here is the ratio between the motions on top of the soil (point A) and on the rock outcrop (point B).

When the acceleration at B is a harmonic motion of frequency *f* (cps) and amplitude a_B, the acceleration at A is also harmonic of the same frequency and amplitude a_A. The amplification ratio a_A/a_B is a function of the ratio of frequencies $f/(v_s/4h)$, of the soil damping b_s, and of the rock/soil impedance ratio which is equal to $g_r v_r / g_s v_s$. Figure C4.1.2-6 presents a_A/a_B calculated for a layer with h = 100 ft (30.5 m), $v_s/4h$ = 1.88 cps, and *IR* = 6.7 (Roesset, 1977). The maximum amplification occurs essentially at the natural frequency of the layer, $f_{soil} = V_s/4h$, and is approximately equal to:

$$\left(\frac{a_A}{a_B}\right)_{max} = \frac{1}{\left(\frac{1}{IR}\right) + \left(\frac{\pi}{2}\right)\beta_s} \qquad \text{(C4.1.2-1)}$$

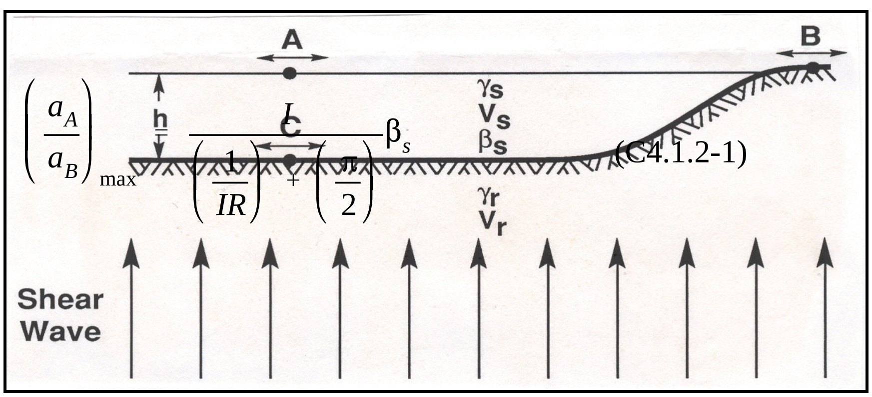


FIGURE C4.1.2.5 Uniform soil layer on elastic rock subjected to vertical shear waves.

That is, the maximum soil/rock amplification for steady-state harmonic motion in this simple model depends on two factors--b_s and *IR*. When *IR* = ¥ (rigid rock), the only way the system can dissipate energy is in the soil and $(a_A/a_B)_{max} = 2/pb_s$ can be very large. For example, if *IR* = ¥ and b_s = 0.04,

$(a_A/a_B)_{max}$ = 16. If *IR* decreases, the amplification $(a_A/a_B)_{max}$ also decreases. For example, if *IR* = 15 and b_s = 0.04, the amplification is cut in half, $(a_A/a_B)_{max}$ = 8.

Another way of expressing the contribution of the impedance ratio *IR* in Eq. C4.1.2-1 is as an "additional equivalent soil damping" with a total damping b_{tot} in the system at its natural frequency:

$$\beta_{tot} \approx \beta_s + \left(\frac{2}{\pi IR} \right) \qquad \text{(C4.1.2-2)}$$

Eq. C4.1.2-2 is very important since the maximum amplification $(a_A/a_B)_{max}$ is always inversely proportional to b_{tot}, not only for the case of the uniform layer but also for other soil profiles on rock. b_{tot} always includes an internal damping contribution (b_s) and a second term reflecting the rock-soil impedance contrast *IR* although the specific definition of *IR* and the numerical factor $2/p$ generally will change depending on the profile. When a soft layer lies on top of a significant thickness of stiffer soil followed by rock, Eq. C4.1.2-2 is still qualitatively valid, but the calculations are more complicated. In that case, the impedance contrast must consider the whole soil profile and, thus, both soft and stiff soils play a role in determining b_{tot} and $(a_A/a_B)_{max}$. Also, the maximum amplification may occur at the natural frequency of the soft layer, of the whole profile, or at some other frequency.

Two-Factor Approach and the 1992 Site Response Workshop: The recommendations developed during the NCEER/SEAOC/BSSC Site Response Workshop mentioned above were summarized by Rinne and Dobry (1992) and are reprinted as Appendix F of this commentary to provide the reader with a better understanding of the thinking behind the current *Provisions*. Some additional background information taken mostly from the proceedings of that workshop (Martin, 1994) is included below.

As discussed above, soil sites generally amplify more the rock spectral accelerations at long periods than at short periods and, for a severe level of shaking (S_S >> 1.0g; S_1 >> 0.4g), the short-period amplification or deamplification is small; this was the basis for the use in the previous versions of the *Provisions*. However, the evidence that short-period accelerations including the peak acceleration can be amplified several times, especially at soft sites subjected to low levels of shaking, suggested the replacement of the normalized spectrum approach by the two-factor approach sketched in Figure C4.1.2-7. In this approach, adopted in the 1994 *Provisions*, the short-period plateau, represented by S_{MS}, is multiplied by a short-period site coefficient F_a and the long period curve represented by S_{M1}/T is multiplied by a long-period site coefficient F_v. Both F_a and F_v depend on the site conditions and on the level of shaking, defined respectively by the values of S_S and S_1.

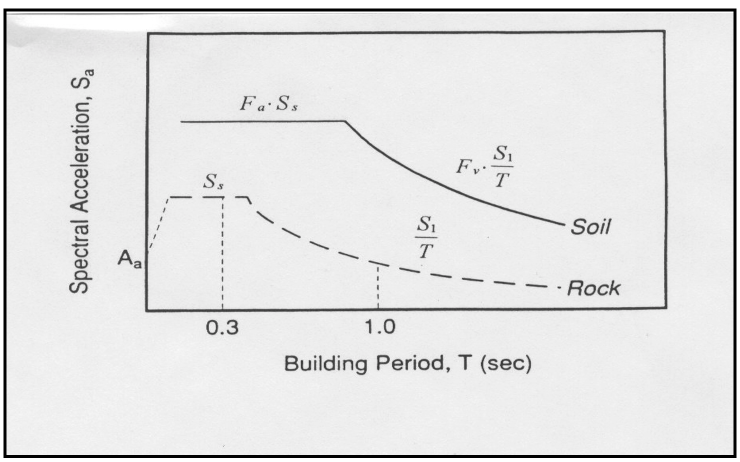


FIGURE C4.1.2.7 Two-factor approach to local site response.

Strong-motion recordings, as obtained from the Loma Prieta earthquake of October 17, 1989, provide important quantitative measures of the *in situ* response of a variety of geologic deposits to damaging levels of shaking. Average amplification factors derived from these data with respect to "firm to hard rock" for short-period (0.1-0.5 sec), intermediate-period (0.5-1.5 sec), mid-period (0.4-2.0 sec), and long-period (1.5-5.0 sec) bands show that a short- and mid-period factor are sufficient to characterize the response of the local site conditions (Borcherdt, 1994). This important result is consistent with the two-factor approach summarized in Figure C4.1.2-7. Empirical regression curves fit to these amplification data as a function of mean shear wave velocity at the site are shown in Figure C4.1.2-8.

These curves provide empirical estimates of the site coefficients F_a and F_v as a function of mean shear wave velocity for input ground motion levels near 0.1g (Borcherdt and Glassmoyer, 1993). The empirical amplification factors predicted by these curves are in good agreement with those derived independently based on numerical modeling of the Loma Prieta strong-motion data (Seed et al., 1992) and those derived from parametric studies of several hundred soil profiles (Dobry et al., 1994b). These empirical relations are consistent with theory in that they imply that the average amplification at a site increases as the rock/soil impedance ratio (IR) increases, similar to the trend described by Eq. C4.1.2-1. They also are consistent with observed correlations between amplification and shear velocity for soft clays in Mexico City (Ordaz and Arciniegas, 1992). These short- and mid-period amplification factors implied by the Loma Prieta strong-motion data and related calculations for the same earthquake by Joyner et al. (1994) as well as modeling results at the 0.1g level provided the basis for the consensus values provided in Tables 4.1.2a and 4.1.2b. Values at higher levels were initially determined from modeling results for soft clays derived by Seed (1994) with values for intermediate soil conditions derived by linear extrapolation. A rigorous framework for extrapolation of the Loma Prieta results consistent with the results in Tables C4.1.2a and C4.1.2b is given in the following paragraph.

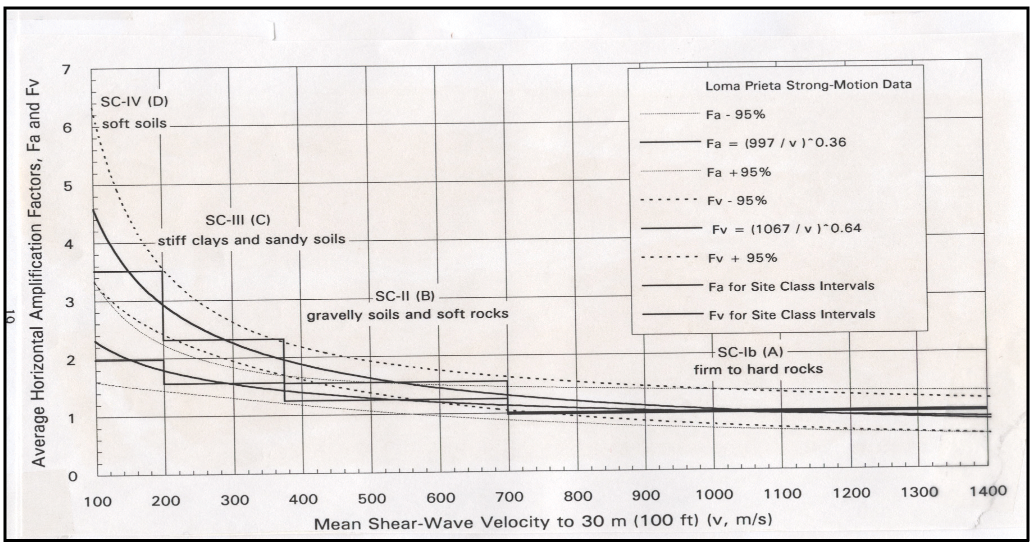
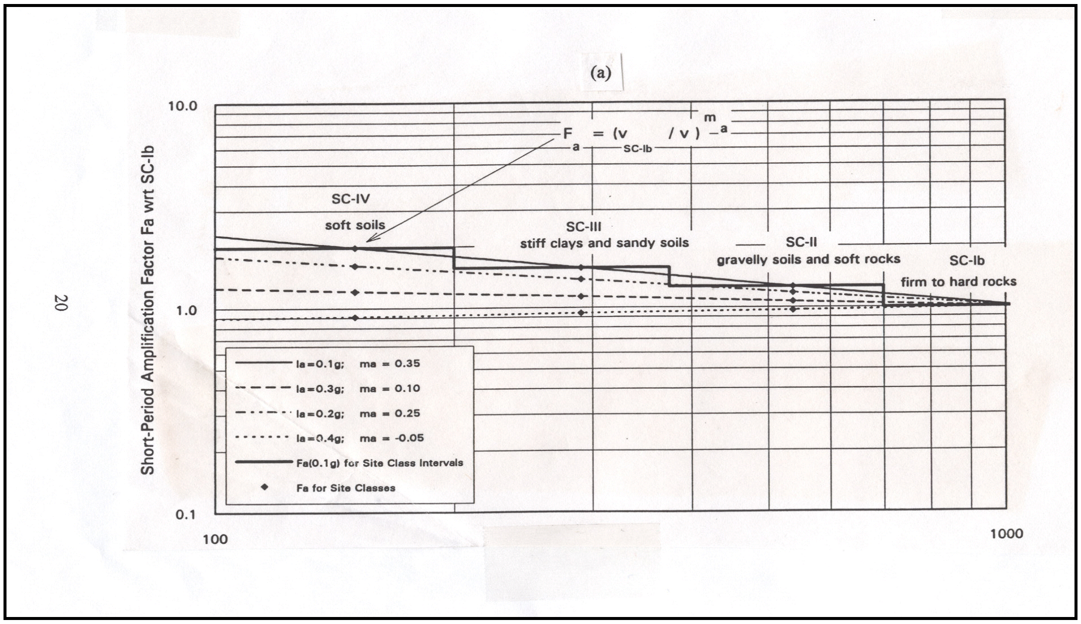


FIGURE C4.1.2-8 Short period F_a and mid-period F_v amplification factors with respect to "firm to hard" rock plotted as a continuous function of mean shear wave velocity using the regression equations derived from the strong-motion recordings of the Loma Prieta earthquake. The 95 percent confidence intervals for the ordinate to the true population regression line and the amplification factors for the simplified site classes also are shown (Borcherdt, 1994).

Extrapolation of amplification estimates at the 0.1g level as derived from the Loma Prieta earthquake must necessarily be based on laboratory and theoretical modeling considerations because few or no strong-motion recordings have been obtained at higher levels of motion, especially on soft soil deposits. Resulting estimates should be consistent with other relations between large rock and soil motions and local site conditions as summarized in Figure C4.1.2-1. The form of the regression curve in Figure C4.1.2-8 suggests a simple and well defined procedure for extrapolation. It shows that the functional relationship between the logarithms of amplification and mean shear velocity is a straight line (Borcherdt, 1993). Consequently, as the amplification factor for "firm to hard" rock is necessarily unity, the extrapolation problem is determined by specification of the amplification factors at successively higher levels of motion for the soft-soil site class. For input ground motion levels near 0.1g, Borcherdt (1993) began with amplification levels specified by the empirical regression curves (Figure C4.1.2-8) for the Loma Prieta strong-motion data. Higher levels of motion were inferred from laboratory and numerical modeling results (Seed et al., 1992; Dobry et al., 1994a). The resulting short-period (F_a) and mid-period (F_v) site coefficients as a function of mean shear velocity (v--labeled $_s$ elsewhere in this *Commentary* and in the *Provisions*) and input ground motion level (I_a) specified with respect to "firm to hard" rock are given in Figure C4.1.2-9 and plotted with logarithmic scales. These expressions state that the average amplification at a site is equal to the "rock-soil" impedance ratio raised to an exponent (*ma* or *mv*). These exponents are defined as 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 at the specified input ground motion level (Borcherdt, 1993). The equations in Figure C4.1.2-9 provide a framework to illustrate a simple procedure for derivation of amplification factors that are in general agreement with the consensus values included in Tables 1.4.2.3a and 1.4.2.3b of the *Provisions*. However, the numbers in these tables of the *Provisions* are not necessarily identical to the equations' predictions due to other considerations discussed during the consensus process.

Extensive site response studies using both equivalent linear and nonlinear programs were conducted by several groups as listed by Rinne and Dobry (1992). The main objectives of these studies were to generalize the experience of well documented earthquakes such as Loma Prieta and Mexico City to a variety of site conditions and earthquake types and levels of shaking. Some results obtained by Dobry et al. (1994a) are reproduced in Figures C4.1.2-10 to C4.1.2-12.

Figure C4.1.2-10 presents values of peak amplification at long periods for soft sites (labeled RRS_{max} in the figure) calculated using the equivalent linear approach as a function of the plasticity index (PI) of the soil, rock wave velocity v_r, and for weak and strong shaking. The effect of PI is due to the fact that soils with higher PI exhibit less stress-strain nonlinearity and a lower damping b_s (Vucetic and Dobry, 1991). For $S_S A_a$ = 0.25g, $S_!$ = 0.1g, v_r = 4,000 ft/sec (1220 m/s) and PI = 50, roughly representative of Bay area soft sites in the Loma Prieta earthquake, RRS_{max} = 4.4, which coincides with the upper part of the range backfigured by Borcherdt from the records. Note the reduction of this value of RRS_{max} from 4.4 to about 3.3 when S_S = 1.0g, $S_!$ = 0.4g due to soil nonlinearity. Evidence such as this is used in the 1994 *Provisions* to extrapolate values of F_a and F_v at low levels of shaking--based on both analysis and observations--to high levels of shaking for which no observations on soft sites currently are available.



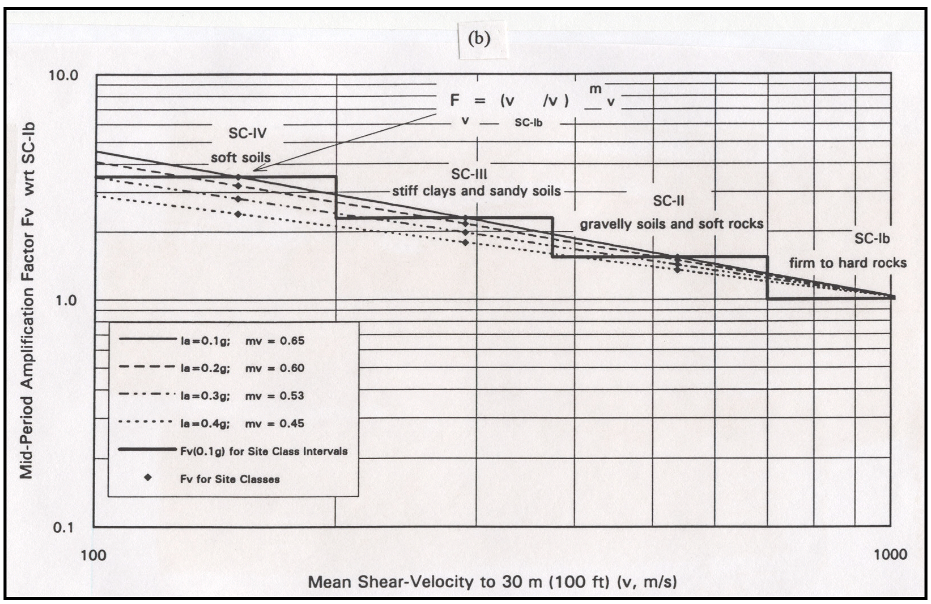


FIGURE C4.1.2-9(a) short-period F_a and (b) mid-period F_v amplification factors with respect to "firm to hard" rock (SC-Ib) plotted with logarithmic scales as a continuous function of mean shear wave velocity using the indicated equations for specified levels of input ground motion. The equations correspond to straight lines determined by the points defined as the logarithms of the amplification factors and shear velocities for the "soft-soil" and "firm to hard" rock site classes. The amplification factors for the "soft-soil" site class are based on strong motion recordings at the 0.1g level and on numerical modeling and expert opinion results for higher levels of motion. The exponents *ma* and *mv* are given by the slope of the indicated straight lines. Amplification factors with respect to SC-Ib for the amplified site classes are shown for the corresponding mean shear wave velocity interval for input ground motion levels near 0.1g (Borcherdt, 1993)

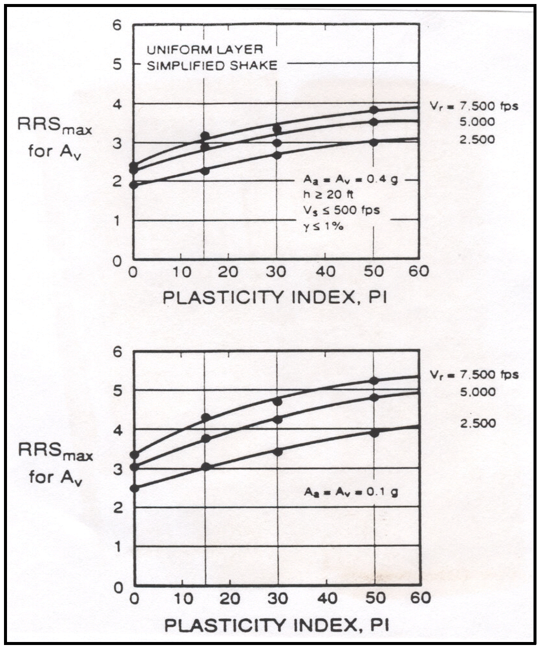


FIGURE C4.1.2-10 Summary of uniform layer analysis using simple SHAKE (Dobry et al., 1994a).

Specific equivalent linear runs using the SHAKE program corresponding to the same situation are included in Figure C4.1.2-11 while Figure C4.1.2-12 summarizes and compares them with calculations by Joyner et al. (1994) from the Loma Prieta records on soft sites similar to the work by Borcherdt mentioned above.

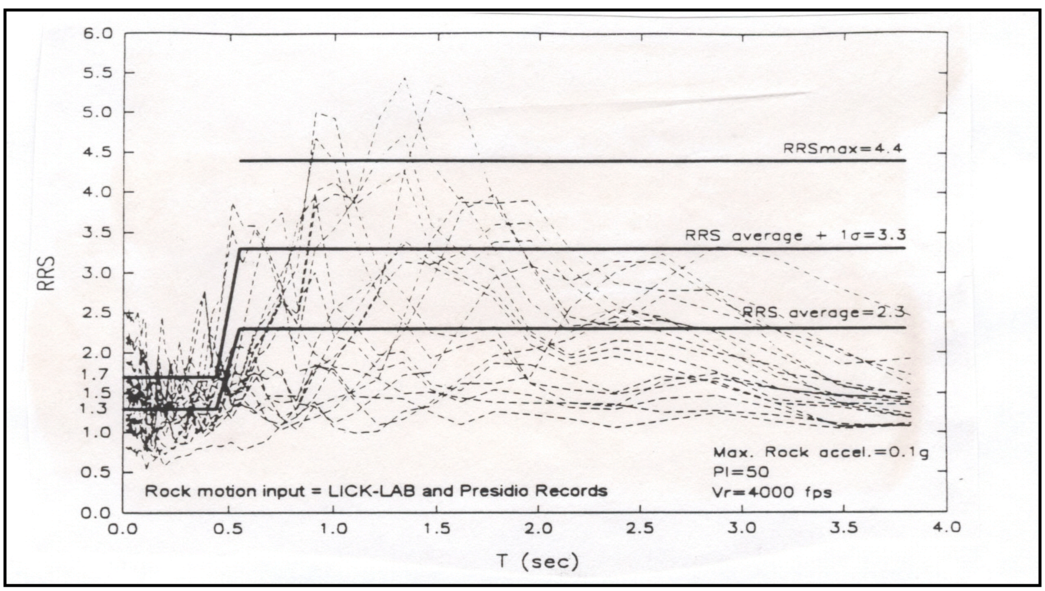


FIGURE C4.1.2-11 Summary of uniform layer analysis usinf SHAKE program, *h*≥50 ft (15.2m) (Dobry et al., 1994a).

Another important observation from analytical results such as shown in Figure C4.1.2-11 is that the values of RRS_{max} are about 20 percent higher for soft sites on "hard rock"--characterized by v_r = 7,500 ft/sec (2290 m/s)--than for soft sites on "regular rock" corresponding to v_r = 4,000 ft/sec (1220 m/s). This is again the impedance ratio effect previously discussed. Separate studies indicate that earthquake motions on outcrops of "hard rock" tend to be smaller than on outcrops of "regular rock" by 10 to 40 percent at both short and long periods (except at very small periods under about 0.2 sec where the reverse may be true); see Su et al. (1992) and Silva (1992). On the basis of these studies and observations, the 1994 *Provisions* incorporate the difference between "regular" rock (B) and "hard" rock of $_s$ > 5,000 ft/sec (1520 m/s) by defining a new "hard rock" site category (A) and assigning to it site factors $F_a = F_v = 0.8$.

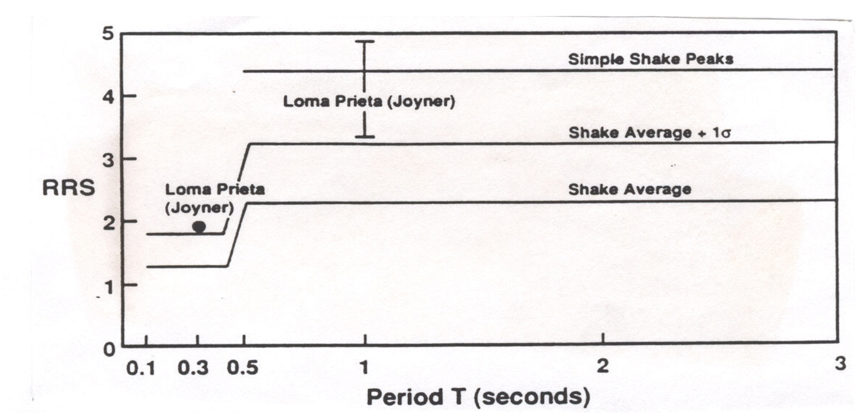


FIGURE 4.1.2-12 Comparison between RRS SHAKE program results and those obtained by Joyner et al. (1994) for the 1989 Loma Prieta event (Dobry et al., 1994a).

Use of Geotechnical Parameters Instead of v_s: Based on the studies and observations discussed above, the site categories in the 1994 *Provisions* are defined in terms of the average shear wave velocity in the top 100 ft (30.5 m) of the profile, v_s. If the shear wave velocities are available for the site, they should be used.

However, in recognition of the fact that in many cases the shear wave velocities are not available, alternative definitions of the site categories also are included in the 1994 *Provisions*. They use the standard penetration resistance for cohesionless soil layers and the undrained shear strength for cohesive soil layers. These alternative definitions are rather conservative since the correlation between site amplification and these geotechnical parameters is more uncertain than that with v_s. That is, there will be cases when the values of F_a and F_v will be smaller if the site category is based on v_s rather than on the geotechnical parameters. Also, the reader must not interpret the site category definitions as implying any specific numerical correlation between shear wave velocity on the one hand and standard penetration or shear strength on the other.

Conducting Site-Specific Geotechnical Investigations and Dynamic Site Response Analysis for Site Class F Soils: As indicated in Sec. 4.1.2.1 and in notes to Tables 4.1.2.4a and b, site coefficients F_a and F_v are not provided for Site Class F soils and site-specific geotechnical investigations and dynamic site response analyses are required for these soils. The exception is that for structures having a fundamental period of vibration equal to or less than 0.5 second, values of F_a and F_v for liquefiable soils, may be determined by following the steps for classifying a site in Sec. 4.1.2.2 assuming liquefaction does not occur. The exception is provided because ground motion data obtained in liquefied soil areas during earthquakes indicate that short-period ground motions are attenuated due to liquefaction whereas long-period ground motions may be amplified. Guidelines are provided below for conducting site-specific investigations and site response analyses for Site Class F soils. These guidelines are also applicable if it is desired to conduct dynamic site response analyses for other soil ypes.

Site-Specific Geotechnical Investigation: For purposes of obtaining data to conduct a site response analysis, site-specific geotechnical investigations should include borings with sampling, standard penetration tests (SPTs), cone penetrometer tests (CPTs), and/or other subsurface investigative techniques and laboratory soil testing to establish the soil types, properties, and layering and the depth to rock or rock-like material. It is desirable to measure shear wave velocities in all soil layers. Alternatively, shear wave velocities may be estimated based on shear wave velocity data available for similar soils in the local area or through correlations with soil types and properties. A number of such correlations are summarized by Kramer (1996).

Dynamic Site Response Analysis: *Components* of a dynamic site response analysis include the following steps:

1. Modeling the soil profile--Typically, a one-dimensional soil column extending from the ground surface to bedrock is adequate to capture first-order site response characteristics. However, two- to three-dimensional models may be considered for critical projects when two or three-dimensional wave propagation effects may be significant (e.g., in basins). The soil layers in a one-dimensional model are characterized by their total unit weights shear wave velocities from which low-strain (maximum) shear moduli may be obtained, and by relationships defining the nonlinear shear stress-strain relationships of the soils. The required relationships for analysis are often in the form of curves that describe the variation of shear modulus with shear strain (modulus reduction curves) and by curves that describe the variation of damping with shear strain (damping curves). In a two- or three-dimensional model, compression wave velocities or moduli or Poissons ratios also are required. In an analysis to estimate the effects of liquefaction on soil site response, the nonlinear soil model also must incorporate the buildup of soil pore water pressures and the consequent effects on reducing soil stiffness and strength. Typically, modulus reduction curves and damping curves are selected on the basis of published relationships for similar soils (e.g., Seed and Idriss, 1970; Seed et al., 1986; Sun et al., 1988; Vucetic and Dobry, 1991; Electric Power Research Institute, 1993; Kramer, 1996). Site-specific laboratory dynamic tests on soil samples to establish nonlinear soil characteristics can be considered where published relationships are judged to be inadequate for the types of soils present at the site. The uncertainty in soil properties should be estimated, especially the uncertainty in the selected maximum shear moduli and modulus reduction and damping curves.

2. Selecting input rock motions-- Acceleration time histories that are representative of horizontal rock motions at the site are required as input to the soil model. Unless a site-specific analysis is carried out to develop the rock response spectrum at the site, the maximum considered earthquake (MCE) rock spectrum for Site Class B rock can be defined using the general procedure described in Sec. 4.1.2. For hard rock (Site Class A), the spectrum may be adjusted using the site factors in Tables 4.1.2.4a and b. For profiles having great depths of soil above Site Slass A or B rock, consideration can be given to defining the base of the soil profile and the input rock motions at a depth at which soft rock or very stiff soil of Site Class C is encountered. In such cases, the MCE rock response spectrum may be taken as the spectrum for Site Class C defined using the site factors in Tables 4.1.2.4a and b. Several acceleration time histories, typically at least four, recorded during earthquakes having magnitudes and distances that significantly contribute to the site seismic hazard should be selected for analysis. The U.S. Geological Survey results for deaggregation of seismic hazard (website address: http://geohazards.cr.usgs.gov/eq/) can be used to evaluate the dominant magnitudes and distances contributing to the hazard. Prior to analysis, each time history should be scaled so that its spectrum is at the approximate level of the MCE rock response spectrum in the period range of interest. It is desirable that the average of the response spectra of the suite of scaled input time histories be approximately at the level of the MCE rock response spectrum in the period range of interest. Because rock response spectra are defined at the ground surface rather than at depth below a soil deposit, the rock time histories should be input in the analysis as outcropping rock motions rather than at the soil-rock interface.

3. Site response analysis and results interpretation-- Analytical methods may be equivalent linear or nonlinear. Frequently used computer programs for one-dimensional analysis include the equivalent linear program SHAKE (Idriss and Sun, 1992) and the nonlinear programs DESRA-2 (Lee and Finn, 1978), MARDES (Chang et al., 1991), SUMDES (Li et al., 1992), D-MOD (Matasovic, 1993), and TESS (Pyke, 1992). For analysis of liquefaction effects on site response, computer programs incorporating pore water pressure development (effective stress analyses) must be used (e.g., DESRA-2, SUMDES, D-MOD, and TESS). Response spectra of output motions at the ground surface should be calculated and the ratios of response spectra of ground surface motions to input outcropping rock motions should be calculated. Typically, an average of the response spectral ratio curves is obtained and multiplied by the MCE rock response spectrum to obtain the MCE soil design response spectrum. Sensitivity analyses to evaluate effects of soil property uncertainties should be conducted and considered in developing the design response spectrum.

4.1.2.5 Design Spectral Response Acceleration Parameters: This section provides a general method for obtaining a 5 percent damped response spectrum from the site design acceleration response parameters S_{aS} and S_{a1}. This spectrum is based on that proposed by Newmark and Hall, as a series of three curves representing in the short period, a region of constant spectral response acceleration; in the long period a range of constant spectral response velocity; and in the very long period, a range of constant spectral response displacement. Response acceleration at any period in the long period range can be related to the constant response velocity by the equation:

$$S_a = \omega S_v = \frac{2\pi}{T} S_v \qquad \text{(C4.1.2.5-1)}$$

where ω is the circular frequency of motion, T is the period and S_v is the constant spectral response velocity. The site design spectral response acceleration at 1 second, S_{a1}, therefore is simply related to the constant spectral velocity for the spectrum by the relation:

$$S_{a1} = 2\pi S_v \qquad \text{(C4.1.2.5-2)}$$

and the spectral response acceleration at any period in the constant velocity range can be obtained from the relationship:

$$S_a = \frac{S_{a1}}{T} \qquad \text{(C4.1.2.5-3)}$$

The constant displacement domain of the response spectrum is not included on the generalized response spectrum because relatively few structures have a period long enough to fall into this range. Response accelerations in the constant displacement domain can be related to the constant displacement by a $1/T^2$ relationship. Sec. 5.5 of the *Provisions,* which provides the requirements for modal analysis also provides instructions for obtaining response accelerations in the very long period range.

4.2 SEISMIC DESIGN CATEGORY: This section establishes the five design categories that are the keys for establishing design requirements for any building based on its use (Seismic Use Group) and on the level of expected seismic ground motion. Once the Seismic Design Category (A, B, C, D, E, or F) for the building is established, many other requirements such as detailing, quality assurance, systems and height limitations, specialized requirements, and change of use are related to it.

Prior to the 1997 edition of the *Provisions*, these categories were termed Seismic Performance Categories. While the desired performance of the building, under the design earthquake, was one consideration used to determine which category a building should be assigned to, it was not the only factor. The seismic hazard at the site was actually the principle parameter that affected a building's category. The name was changed to Seismic Design Category to represent the uses of these categories, which is to determine the specific design requirements.

The earlier editions of the *Provisions* utilized the peak velocity related acceleration, A_v, to determine a building's Seismic Performance Category. However, this coefficient does not adequately represent the damage potential of earthquakes on sites with soil conditions other than rock. Consequently, the 1997 *Provisions* adopted the use of response spectral acceleration parameters S_{DS} and S_{D1}, which include site soil effects for this purpose. Instead of a single table, as was present in previous editions of the *Provisions*, two tables are now provided, relating respectively to short period and long period structures.

Seismic Design Category A represents structures in regions where anticipated ground motions are minor, even for very long return periods. For such structures, the *Provisions* require only that a complete lateral-force-resisting system be provided and that all elements of the structure be tied together. A nominal design force of 1 percent of the weight of the structure is used to proportion the lateral system.

It is not considered necessary to specify seismic-resistant design on the basis of a maximum considered earthquake ground motion for Seismic Design Category A structures because the ground motion computed for the areas where these structures are located is determined more by the rarity of the event with respect to the chosen level of probability than by the level of motion that would occur if a small but close earthquake actually did occur. However, it is desirable to provide some protection against both earthquakes and many other types of unanticipated loadings. Thus, the requirements for Seismic Design Category A provide a nominal amount of structural integrity that will improve the performance of buildings in the event of a possible but rare earthquake even though it is possible that the ground motions could be large enough to cause serious damage or even collapse. The result of design to Seismic Design Category A requirements is that fewer building would collapse in the vicinity of such an earthquake.

The integrity is provided by a combination of requirements. First, a complete load path for lateral forces must be identified. Then it must be designed for a lateral force equal to a 1 percent acceleration on the mass. The minimum connection forces specified for Seismic Design Category A also must be satisfied.

The 1 percent value has been used in other countries as a minimum value for structural integrity. For many structures, design for the wind loadings specified in the local buildings codes normally will control the lateral force design when compared to the minimum integrity force on the structure. However, many low-rise, heavy structures or structures with significant dead loads resulting from heavy equipment may be controlled by the nominal 1 percent acceleration. Also, minimum connection forces may exceed structural forces due to wind in some structures.

Seismic Design Category B includes Seismic Use Group I and II structures is regions of seismicity where only moderately destructive ground shaking is anticipated. In addition to the requirements for Seismic Design Category A, structures in Seismic Design Category B must be designed for forces determined using Maps 1 through 24.

Seismic Design Category C includes Seismic Use Group III structures in regions where moderately destructive ground shaking may occur as well as Seismic Use Group I and II structures in regions with somewhat more severe ground shaking potential. In Seismic Design Category C, the use of some structural systems is limited and some nonstructural *components* must be specifically design for seismic resistance.

Seismic Design Category D includes structures of Seismic Use Group I, II, and III located in regions expected to experience destructive ground shaking but not located very near major active faults. In Seismic Design Category D, severe limits are placed on the use of some structural systems and irregular structures must be subjected to dynamic analysis techniques as part of the design process.

Seismic Design Category E includes Seismic Use Group I and II structures in regions located very close to major active faults and Seismic Design Category F includes Seismic Use Group III structures in these locations. Very severe limitations on systems, irregularities, and design methods are specified for Seismic Design Categories E and F. For the purpose of determining if a structure is located in a region that is very close to a major active fault, the *Provisions* use a trigger of a mapped maximum considered earthquake spectral response acceleration at 1 second periods, S_1, of 0.75g or more regardless of the structure's fundamental period. The mapped short period acceleration, S_S, was not used for this purpose because short period response accelerations do not tend to be affected by near-source conditions as strongly as do response accelerations at longer periods.

Local or regional jurisdictions enforcing building regulations need to consider the effect of the maps, typical soil conditions, and Seismic Design Categories on the practices in their jurisdictional areas. For reasons of uniformity of practice or reduction of potential errors, adopting ordinances could stipulate particular values of ground motion, particular Site Classes, or particular Seismic Design Categories for all or part of the area of their jurisdiction. For example:

1. An area with an historical practice of high seismic zone detailing might mandate a minimum Seismic Design Category of D regardless of ground motion or Site Class.

2. A jurisdiction with low variation in ground motion across the area might stipulate particular values of the ground motion rather than requiring use of the maps.

3. An area with unusual soils might require use of a particular Site Class unless a geotechnical investigation proves a better Site Class.

4.2.2 Site Limitation for Seismic Design Categories E and F: The forces that result on a structure located astride the trace of a fault rupture that propagates to the surface are extremely large and it is not possibly to reliably design a structure to resist such forces. Consequently, the requirements of this section limit the construction of buildings in Seismic Design Categories E and F on sites subject this hazard. Similarly, the effects of landsliding, liquefaction, and lateral spreading can be highly damaging to a building. However, the effects of these site phenomena can more readily be mitigated through the incorporation of appropriate design measures than can direct ground fault rupture. Consequently, construction on sites with these hazards is permitted, if appropriate mitigation measures are included in the design.

REFERENCES

Borcherdt, R. D. 1994. "Simplified Site Classes and Empirical Amplification Factors for Site-Dependent Code Provisions." In *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.

Borcherdt, R. D. 1993. "General Methodology for Estimating Free-Field Site Specific Response Spectra." In Minutes of the Building Seismic Safety Council meeting, April 5-6, 1993, San Francisco, California.

Borcherdt, R. D., and G. Glassmoyer. 1993. "Influence of Local Geology and Weak Ground Motions in the San Francisco Bay Region, California, and Their Implications for Site-Specific Code Provisions." In *The Loma Prieta Earthquake of October 17, 1989 - Strong Ground Motion*, USGS Professional Paper 1551-A, edited by R. D. Borcherdt.

Chang, C.-Y., C.M. Mok,, M.S. Power, and Y.K. Tang, 1991. *Analysis of Ground Response at Lotung Large-Scale Soil-Structure Interaction Experiment Site*, Report NP-7306-SL. Palo Alto, California: Electric Power Research Institute.

Dobry, R., G. M. Martin, E. Parra, and A. Bhattacharyya. 1994a. "Development of Site-Dependent Ratios of Elastic Response Spectra (RRS) and Site Categories for Building Seismic Codes." In *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.

Dobry, R., G. M. Martin, E. Parra, and A. Bhattacharyya. 1994b. *Study of Ratios of Response Spectra Soil/Rock and of Site Categories for Seismic Codes*. Buffalo, New York: National Center for Earthquake Engineering Research.

Electric Power Research Institute. 1993. *Guidelines for Determining Design Basis Ground Motions*, Report EPRI TR-102293. Palo Alto, California.

Hayashi, S., Tsuchida, and E. Kurata. 1971. "Average Response Spectra for Various Subsoil Conditions." Paper presented at the Third Joint Meeting of the U.S.-Japan Panel on Wind and Seismic Effects, UNJR, Tokyo, May 10-12.

Housner, G. W. 1990. *Competing Against Time: Report to Governor George Deukmejian from the Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake*.

Idriss, I. M. 1990a. "Response of Soft Soil Sites During Earthquakes." In *Proceedings of the Symposium to Honor Professor H. B. Seed*, Berkeley.

Idriss, I. M. 1990b. "Influence of Local Site Conditions on Earthquake Ground Motions." In *Proceedings of the 4th U.S. National Conference on Earthquake Engineering*, Palm Springs, Vol. 1, pp. 55-57.

Idriss, I. M., and J. I.Sun.1992. *User's Manual for SHAKE91*. Davis: Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California.

Jacob, K. 1990. "Seismic Hazards and the Effects of Soils on Ground Motions for the Greater New York City Metropolitan Region." In *Geotechnical Aspects of Seismic Design in the N.Y.C. Metropolitan Areas; Risk Assessment, Code Requirements and Design Techniques*, ASCE Metropolitan Section, page 24, New York, New York, November 13-14.

Jacob, K. 1991. "Seismic Zonation and Site Response: Are Building-Code Soil Factors Adequate to Account for Variability of Soil Conditions Across the US?" In *Proceedings of the 4th International Conference on Seismic Zonation*, Vol. II, pp. 695-702. Oakland, California: Earthquake Engineering Research Institute.

Joyner, W. B., T. E. Fumal, and G. Glassmoyer. 1994. "Empirical Spectral Response Ratios for Strong Motion Data from the 1989 Loma Prieta, California, Earthquake." In *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.

Kramer, S. L., 1996. *Geotechnical Earthquake Engineering*. New Jersey: Prentice Hall.

Lee, M. K. W., and W. D. L. Finn. 1978. *DESRA-2, Dynamic Effective Stress Response Analysis of Soil Deposits with Energy Transmitting Boundary Including Assessment of Liquefaction Potential, Soil Mechanics Series 36*. Vancouver, Canada: Department of Civil Engineering, University of British Columbia.

Li, X. S., Z. L. Wang, and C. K. Shen, 1992. *SUMDES, A Nonlinear Procedure for Response Analysis of Horizontally-Layered Sites Subjected to Multi-Directional Earthquake Loading*. Davis: Department of Civil Engineering, University of California.

Martin, G. M., Editor. 1994. *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20.

Matasovic, N. 1993. Seismic Response of Composite Horizontally-Layered Soil Deposits, Ph.D. dissertation, Civil and Environmental Engineering Department, University of California, Los Angeles.

Milne, W. G., and A. G. Davenport. 1969. "Distribution of Earthquake Risk in Canada." *Bulletin of the Seismological Society of America* 59(2):754-779.

Mohraz, B. 1976. "A Study of Earthquake Response Spectra for Different Geological Conditions." *Bulletin of the Seismological Society of America* 66 (3):915-935.

Newmark, N. M. 1973. *A Study of Vertical and Horizontal Spectra,* Report WASH-1255. Washington, D.C.: U.S. Atomic Energy Commission, Directorate of Licensing.

Ordaz, M., and Arciniegas. 1992. Personal communication to Ricardo Dobry.

Pyke, R. M. 1992, *TESS: A Computer Program for Nonlinear Ground Response Analyses*. Lafayette, California: TAGA Engineering Systems and Software.

Rinne, E., and R. Dobry. 1992. Preliminary Site Recommendations, Memorandum to Roland Sharpe, Chairman TS 2, Building Seismic Safety Council, December 11.

Roesset, J. M. 1977. "Soil Amplification of Earthquakes." In *Numerical Methods in Geotechnical Engineering,* edited by C. S. Desai and J. T. Christian, Chapter 19, pp. 639-682. New York: McGraw-Hill.

Seed, R. B. 1994. In *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions,* University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.

Seed, R. B., S. E. Dickenson, G. A. Rau, R. K. White, and C. M. Mok. 1992. "Observations Regarding Seismic Response Analyses for Soft and Deep Clay Sites."

Seed, R., S. E. Dickenson, M. F. Riemer, J. D. Bray, N. Sitar, J. K. Mitchell, I. M. Idriss, R. E. Kayen, A. Kropp, L. F. Harder Jr., and M. S. Power. 1990. *Preliminary Report on the Principal Geotechnical Aspects of the October 17, 1989, Loma Prieta Earthquake*. Report UCB/EERC-90/05. Berkeley, California: EERC.

Seed, H. B., R. Murarka, J. Lysmer, and I. M. Idriss. 1976a. "Relationships Between Maximum Acceleration, Maximum Velocity, Distance from Source and Local Site Conditions for Moderately Strong Earthquakes." *Bulletin of the Seismological Society of America* 66 (4):1323--1342.

Seed, H. B., M. P. Romo, J. I. Sun, A. Jaime, and J. Lysmer. 1988. "The Mexico Earthquake of September 19, 1985--Relationships Between Soil Conditions and Earthquake Ground Motions." *Earthquake Spectra* 4(4):687-729.

Seed, H. B., C. Ugas, and J. Lysmer. 1976b. "Site Dependent Spectra for Earthquake-Resistant Design." *Bulletin of the Seismological Society of America* 66 (1):221-244.

Seed, H. B., C. Ugas, and J. Lysmer. 1974. *Site-Dependent Spectra for Earthquake-Resistant Design*, Report EERC 74-12. Berkeley, California: EERC.

Seed, H.B., R. T. Wong, I. M. Idriss, and K. Tokimatsu. 1986. "*Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils*", Journal of Geotechnical Engineering, ASCE, Vol. 112, No. 11: 1016-1032.

Seed, H.B., and I. M. Idriss. 1970, *Soil Moduli and Damping Factors for Dynamic Response Analyses*, Report EERC 70-10. Berkeley: University of California, Earthquake Engineering Research Center.

Silva, W. 1992. Personal communication to Ricardo Dobry.

Structural Engineers Association of California. 1970. *Report of the Ad Hoc Committee on Cost of Design for Earthquakes.* Washington, D.C.: Office of Science and Technology, Task Force on Earthquake Hazard Reduction.

Su, F., K. Aki, T. Teng, Y. Zeng, S. Koyanagi, and K. Mayeda. 1992. "The Relation Between Site Amplification Factor and Surficial Geology in Central California." *Bulletin of the Seismological Society of America* 82(2):580-602.

Sun, J. I., R. Golesorkhi, and H. B. Seed. 1988. *Dynamic Moduli and Damping Ratios for Cohesive Soils*, Report UBC/EERC-88/15. Berkeley: University of California, Earthquake Engineering Research Center.

Vucetic, M., and R. Dobry. 1991. "*Effect of Soil Plasticity on Cyclic Response*", Journal of Geotechnical Engineering, ASCE, Vol. 117, No. 1: 89-107.

Whitman, R., Editor. 1992. *Proceedings of the Site Effects Workshop, October 24-24, 1991*, Report NCEER-92-0006. Buffalo, New York: National Center for Earthquake Engineering Research.