Chapter 3 Commentary

GROUND MOTION

3.1 GENERAL

3.1.3 Definitions. 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. 1.2 of the Provisions establishes a series of Seismic Use Groups, which are used to assign each structure to a specific Seismic Design Category. It is the intent of the Provisions that meeting the seismic design criteria will provide a uniform margin against failure 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 probability 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 correspond to a factor of about 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 probability 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 that the selection of the 2 percent probability of exceedance in 50 years 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 probability of exceedance in 50 years 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 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 to this Commentary volume. The procedure by which these maps were created, as described above and in Appendix A, is
also included in the Provisions under Sec 3.4 so that registered design professionals performing such studies may use methods consistent with those that served as the basis for developing the maps.

3.2 GENERAL REQUIREMENTS

3.2.2 Procedure selection. 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. 3.3. The site-specific procedure is described in Sec. 3.4.

3.3 GENERAL PROCEDURE

This section provides the procedure for obtaining design site spectral response accelerations using the maps provided with the Provisions. Many buildings and structures will be designed using the equivalent lateral force procedure of Sec. 5.2, and this general procedure to determine the design spectral response acceleration parameters, \( S_{DS} \) and \( S_{DI} \), that are directly used in that procedure. Some structures will be designed using the response spectrum procedure of Sec. 5.3. 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 \( S_{DI} \).

Maps 1 and 2 respectively provide two parameters, \( S_5 \) 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. 3.4 provides guidance on site-specific procedures.

\( S_5 \) 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 of 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, \( S_5 \) 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 \( S_5 \) and \( S_1 \) values, as indicated in Sec.3.3.2. This modification is performed with the use of two coefficients, \( F_a \) and \( F_v \), which respectively scale the \( S_5 \) and \( S_1 \) values determined for firm rock sites to values appropriate for other site conditions. The maximum considered earthquake spectral response accelerations adjusted for Site Class effects are designated \( S_{MS} \) and \( S_{MI} \), respectively, 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 \( S_{DI} \), 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 \( S_{MI} \), and completely define a design response spectrum for sites of any characteristics.
Sec. 3.5.1 provides a categorization of the various classes of site conditions, as they affect the design response acceleration parameters. Sec. 3.5.2 describes the steps by which sites can be classified as belonging to one of these Site Classes.

3.3.2 Site coefficients and adjusted acceleration parameters. The site coefficients $F_a$ and $F_v$ presented in Provisions Tables 3.3-1 and 3.3-2 are based on the research described in the following paragraphs.

It has long been recognized that the effects of local soil conditions on ground motion characteristics should be considered in building design. The 1989 Loma Prieta earthquake provided abundant strong motion data that was used extensively together with other information in introducing the site coefficients $F_a$ and $F_v$ into the 1994 Provisions.

The amount of ground motion amplification by a soil deposit relative to bedrock depends on the wave-propagation characteristics of the soil, which can be estimated from measurements or inferences of shear-wave velocity and in turn the shear modulus for the materials as a function of the level of shaking. In general, softer soils with lower shear-wave velocities exhibit higher amplifications than stiffer soils with higher shear velocities. Increased levels of ground shaking result in increased soil stress-strain nonlinearity and increased soil damping which in general reduces the amplification, especially for shorter periods. Furthermore, for soil deposits of sufficient thickness, soil amplification is generally greater at longer periods than at the shorter periods. Based on the studies summarized below, values of the soil amplification factors (site coefficients) shown in Tables 3.3-1 and 3.3-2 were developed as a function of site class and level of ground shaking. Table 3.3-1 presents the short-period site coefficient, $F_a$; Table 3.3-2 presents the long-period site coefficient, $F_v$. As described in Sec. 3.5, Site Classes A through E describe progressively softer (lower shear wave velocity) soils.

Strong-motion recordings obtained on a variety of geologic deposits during the Loma Prieta earthquake of October 17, 1989 provided an important empirical basis for the development of the site coefficients $F_a$ and $F_v$. Figure C3.3.2-1 presents average response spectra of ground motions recorded on soft clay and rock sites in San Francisco and Oakland during the Loma Prieta earthquake. The peak acceleration (which plots at zero-period of the response spectra) was about 0.08 to 0.1 g at the rock sites and was amplified two to three times to 0.2 g or 0.3 g at the soft soil sites. The response spectral accelerations at short periods (~ 0.2 or 0.3 second) were also amplified on average by factors of 2 or 3. It can be seen in Figure C3.3.2-1 that, at longer periods between about 0.5 and 1.5 or 2 seconds, the amplifications of response spectra on the soft clay site relative to rock were even greater, ranging from about 3 to 6 times. Ground motions on stiff soil sites were also observed to be amplified relative to rock sites during the Loma Prieta earthquake, but by smaller factors than on soft soils.
Figure C3.3.2-1. Average spectra recorded during the 1989 Loma Prieta earthquake in San Francisco Bay area at rock sites and soft soil sites (modified after Housner, 1990).

Average amplification factors derived from the Loma Prieta earthquake data with respect to “firm to hard rock” for short-period (0.1-0.5 sec), intermediate-period (0.5-1.4 sec), mid-period (0.4-2.0 sec), and long-period (1.5-5.0 sec) bands, show that a short-period factor and a mid-period factor (the mid-period factor was later renamed the long-period factor in the NEHRP Provisions) are sufficient to characterize the response of the local site conditions (Borcherdt, 1994). This important result is consistent with the two-factor approach to response spectrum construction summarized in Figure C3.3.2-2. Empirical regression curves fitted to these amplification data as a function of mean shear wave velocity at a site are shown in Figure C3.3.2-3.
Figure C3.3.2-2. Two-factor approach to local site response.

Figure C3.3.2-3. Short-period $F_a$ and long-period $F_v$ site coefficients with respect to site class B (firm to hard rocks) inferred as a continuous function of shear-wave velocity from empirical regression curves derived using Loma Prieta strong-motion recordings. The 95 percent confidence intervals for the ordinate to the true population regression line and the corresponding site coefficients in Tables 3.3-1 and 3.3-2 for 0.1g acceleration are plotted. The curves show that a two factor approach with a short- and a long-period site coefficient is needed to characterize the response of near surface deposits (modified from Borcherdt 1994).

The curves in Figure C3.3.2-3 provide empirical estimates of the site coefficients $F_a$ and $F_v$ as a function of mean shear wave velocity for an input peak ground accelerations on rock equal to about 0.1 g (Borcherdt, 1994; Borcherdt and Glassmoyer, 1994) The empirical amplification factors predicted by these curves are in good agreement with those obtained from empirical analyses of Loma Prieta data.
for soft soils by Joyner et al. (1994) shown in Figure 3.3.2-4. These short- and long-period amplification factors for low peak ground (rock) acceleration levels (~ 0.1 g) provided the basis for the values in the left-hand columns of Tables 3.3-1 and 3.3-2. Note that in Tables 3.3-1 and 3.3-2, a peak ground (rock) acceleration of 0.1g corresponds approximately to a response spectral acceleration on rock at 0.2-second period ($S_s$) equal to 0.25g (Table 3.3-1) and to a response spectral acceleration on rock at 1.0-second period ($S_1$) equal to 0.1g (Table 3.3-2).

Figure C3.3.2-4. Calculation of average ratios of response spectra (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).

The values of $F_a$ and $F_v$ obtained directly from the analysis of ground motion records from the Loma Prieta earthquake 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 amplification during earthquakes (e.g., Seed and Idriss 1982), was used extensively for this calibration. Seed et al. (1994) and Dobry et al. (1994) showed that the one-dimensional model provided a good first-order approximation to the observed site response in the Loma Prieta earthquake, especially at soft clay sites. Idriss (1990, 1991) used these analysis techniques to study the amplification of peak ground acceleration on soft soil sites relative to rock sites as a function of the peak acceleration on rock. Results of these studies are shown in Figure 3.3.2-5, illustrating that the large amplifications of peak acceleration on soft soil for low rock accelerations recorded during the 1985 Mexico City earthquake and the 1989 Loma Prieta earthquake should tend to decrease rapidly as rock accelerations increases above about 0.1 g.
After calibration, these 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. Parametric studies involving combinations of hundreds of soil profiles and several dozen input earthquake rock motions provided quantitative guidelines for extrapolation of the Loma Prieta earthquake results (Seed et al. 1994; Dobry et al. 1994). Figure C3.3.2-6 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 (termed maximum Ratio of Response Spectra, $RRS_{\text{max}}$) calculated using the equivalent linear approach as a function of the plasticity index (PI) of the soil and the rock shear wave velocity $V_r$ for both weak (0.1 g) and strong (0.4 g) 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 (Vucetic and Dobry 1991). For peak rock acceleration = 0.1 g, $V_r = 4,000$ ft/sec (1220 m/s) and $PI = 50$, roughly representative of San Francisco Bay area soft sites in the Loma Prieta earthquake, $RRS_{\text{max}} = 4.4$, which for a soil shear wave velocity of 150 m/sec coincides with the upper part of the range in Figure 3.3.2-3 inferred from the ground motion records. Note the reduction of this value of $RRS_{\text{max}}$ from 4.4 to about 3.3 in Figure C3.3.2-6 when peak rock acceleration = 0.4 g, due to soil nonlinearity. Results such as those in Figure C3.3.2-6 provided the basis for the values of $F_a$ and $F_v$ shown in the right-most four columns of Tables 3.3-1 and 3.3-2.
Figure C3.3.2-6. Variation of $R_{RS_{\text{max}}}$ of uniform layer of soft clay on rock from equivalent linear site response analyses (Dobry et al., 1994).

Graphs and equations that provide a framework for extrapolation of $F_a$ and $F_v$ from Loma Prieta results to larger input ground motion levels continuously as a function of site conditions (shear-wave velocity) are shown in Figure C3.3.2-7. The site coefficients in Tables 3.3-1 and 3.3-2 are superimposed on this figure. These simple curves were developed to reproduce the site coefficients for site classes E and B and provide approximate estimates of the coefficients for the other Site Classes at various ground acceleration levels. The equations describing the curves indicate that the amplification at a site is proportional to the shear velocity ratio (impedance ratio) with an exponent that varies with the input ground motion level (Borcherdt, 1994).
Figure C3.3.2-7. Graphs and equations that provide a simple framework for inference of (a) $F_a$ and (b) $F_v$ values as a continuous function of shear velocity at various input acceleration levels. Site coefficients in Table 3.3-1 and 3.3-2 are superimposed. These simple curves were developed to reproduce the site coefficients for site classes E and B and provide approximate estimates of the coefficients for the other site classes at various ground acceleration levels (from Borcherdt 1994).

A more extensive discussion of the development of site coefficients is presented by Dobry, et al. (2000). Since the development of these coefficients and the development of a community consensus regarding their values in 1992, recent earthquakes have provided additional strong motion data from which to infer site amplifications. Analyses conducted on the basis of these more recent data are reported by a number of researchers, including Crouse and McGuire, 1996; Dobry et al., 1999; Silva et al., 2000; Joyner and Boore, 2000; Field, 2000; Steidl, 2000; Rodríguez-Marek et al., 2001; Borcherdt, 2002, and Stewart et al., 2003. While the results of these studies vary, overall the site amplification...
factors are generally consistent with those in Tables 3.3-1 and 3.3.2 and there is no clear consensus for change at the present time (end of 2002).

3.3.4 Design response spectrum. This section provides a general method for obtaining a 5-percent-damped response spectrum from the site design acceleration response parameters $S_a$ and $S_{al}$. 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:

$$S_a = \omega S_v = \frac{2\pi}{T} S_v$$  \hspace{1cm} (C3.3-1)

where $\omega$ is the circular frequency of motion, $T$ is the period, and $S_v$ is the constant spectral response velocity. Thus the site design spectral response acceleration at 1 second, $S_{al}$, is simply related to the constant spectral velocity for the spectrum as follows:

$$S_{al} = 2\pi S_v$$  \hspace{1cm} (C3.3-2)

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

$$S_a = \frac{S_{DL}}{T}$$  \hspace{1cm} (C3.3-3)

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. 5.3 of the Provisions, which provides the requirements for modal analysis also provides instructions for obtaining response accelerations in the very long period range.

The $T_c$ maps were prepared following a two-step procedure. The first step consisted of establishing a correlation between earthquake magnitude and $T_c$. This correlation was established by (1) determining the corner period between intermediate and long period motions based on seismic source theory and (2) examining the response spectra of strong motion accelerograms recorded during moderate and large magnitude earthquakes. This corner period, $T_c$, marks the transition between the constant displacement and constant velocity segments of the Fourier spectrum representing a theoretical fault-rupture displacement history. $T_c$, which was considered an approximation for $T_L$, was related to moment magnitude, $M$, through the formula, $\log T_c = -1.25 + 0.3 M$. This formula was selected from several available formulas based on comparisons of $T_c$ predicted by this equation and $T_L$ estimated from strong motion accelerograms with reliable long period content. The results were used to establish the following half-unit ranges of $M$ for given values of $T_c$.,
<table>
<thead>
<tr>
<th>$M$</th>
<th>$T_c$ (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.0 – 6.5</td>
<td>4</td>
</tr>
<tr>
<td>6.5 – 7.0</td>
<td>6</td>
</tr>
<tr>
<td>7.0 – 7.5</td>
<td>8</td>
</tr>
<tr>
<td>7.5 – 8.0</td>
<td>12</td>
</tr>
<tr>
<td>8.0 – 8.5</td>
<td>16</td>
</tr>
<tr>
<td>8.5 – 9.0+</td>
<td>20</td>
</tr>
</tbody>
</table>

To determine the $T_L$ values for the U.S., the USGS constructed maps of the modal magnitudes ($M_d$) in half-unit increments (as shown in the above table). The maps were prepared from a deaggregation of the 2 percent in 50-yr hazard for $S_a (T = 2$ sec), the response spectral acceleration at an oscillator period of 2 sec. (for HI the deaggregation was only available for $T = 1$ sec). The $M_d$ that was computed represented the magnitude interval that had the largest contribution to the 2 percent in 50-yr hazard for $S_a$.

The $M_d$ maps were judged to be an acceptable approximation to values of $M_d$ that would be obtained if the deaggregation could have been computed at the longer periods of interest. These $M_d$ maps were color coded to more easily permit the eventual construction of the $T_L$ maps. Generally the $T_L$ maps corresponded to the $M_d$ maps, but some smoothing of the boundaries separating $T_L$ regions was necessary to make them more legible. A decision was made to limit the $T_L$ in the broad area in the central and eastern U.S., which had an $M_d$ of 16 sec, to 12 sec. Likewise, the $T_L$ for the areas affected by the great megathrust earthquakes in the Pacific Northwest and Alaska, was limited to 16 sec.

### 3.4 SITE-SPECIFIC PROCEDURE

The objective in conducting a site-specific ground motion analysis is to develop ground motions that are determined with higher confidence for the local seismic and site conditions than can be determined from national ground motion maps and the general procedure of Sec. 3.3. Accordingly, such studies must be comprehensive and incorporate current scientific interpretations. Because there is typically more than one scientifically credible alternative for models and parameter values used to characterize seismic sources and ground motions, it is important to formally incorporate these uncertainties in a site-specific probabilistic analysis. For example, uncertainties may exist in seismic source location, extent and geometry; maximum earthquake magnitude; earthquake recurrence rate; choices for ground motion attenuation relationships; and local site conditions including soil layering and dynamic soil properties as well as possible two- or three-dimensional wave propagation effects. The use of peer review for a site-specific ground motion analysis is encouraged.

Near-fault effects on horizontal response spectra include (1) directivity effects that increase ground motions for periods of vibration greater than approximately 0.5 second for fault rupture propagating toward the site; and (2) directionality effects that increase ground motions for periods greater than approximately 0.5 second in the direction normal (perpendicular) to the strike of the fault. Further discussion of these effects is contained in Somerville et al. (1997) and Abrahamson (2000).

**Conducting site-specific geotechnical investigations and dynamic site response analyses.**

*Provisions* Tables 3.3-1 and 3.3-2 and Sec. 3.5.1 require that site-specific geotechnical investigations and dynamic site response analysis be performed for sites having Site Class F soils. Guidelines are provided below for conducting site-specific investigations and site response analyses for these soils. These guidelines are also applicable if it is desired to conduct dynamic site response analyses for other site classes.

**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) for sandy soils, 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. For very deep soil sites, the depth of investigation need not necessarily extend to bedrock but to a depth that may serve as the location of input motion for a dynamic site response analysis (see below). 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. For very deep soils, the model of the soil columns may extend to very stiff or very dense soils at depth in the column. Two- or three-dimensional models should be considered for critical projects when two or three-dimensional wave propagation effects should be significant (e.g., in basins). The soil layers in a one-dimensional model are characterized by their total unit weights and 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 soil shear modulus with shear strain (modulus reduction curves) and by curves that describe the variation of soil damping with shear strain (damping curves). In a two- or three-dimensional model, compression wave velocities or moduli or Poisson 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. Shear and compression wave velocities and associated maximum moduli should be selected on the basis of field tests to determine these parameters or published relationships and experience for similar soils in the local area. 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. 3.3. For hard rock (Site Class A), the spectrum may be adjusted using the site factors in Tables 3.3-1 and 3.3-2. For profiles having great depths of soil above Site Class 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 3.3-1 and 3.3-2. Several acceleration time histories of rock motions, typically at least four, should be selected for site response analysis. These time histories should be selected after evaluating the types of earthquake sources, magnitudes, and distances that predominantly contribute to the seismic hazard at the site. Preferably, the time histories selected for analysis should have been recorded on geologic materials similar to the site class of materials at the base of the site soil profile during earthquakes of similar types (e.g. with respect to tectonic environment and type of faulting), magnitudes, and distances as those predominantly contributing to the site
seismic hazard. The U.S. Geological Survey national seismic hazard mapping project website (http://geohazards.cr.usgs.gov/eq/) includes hazard deaggregation options and can be used to evaluate the predominant types of earthquake sources, magnitudes, and distances contributing to the hazard. Sources of recorded acceleration time histories include the data bases of the Consortium of Organizations for Strong Motion Observation Systems (COSMOS) Virtual Data Center web site (db.cosmos-eq.org) and the Pacific Earthquake Engineering Research Center (PEER) Strong Motion Data Base website (http://peer.berkeley.edu/smcat/). 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 (Schnabel et al., 1972; 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), TESS (Pyke, 1992), and DESRAMUSC (Qiu, 1998). If the soil response is highly nonlinear (e.g., high acceleration levels and soft soils), nonlinear programs may be preferable to equivalent linear programs. 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, TESS, and DESRAMUSC). 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 a soil response spectrum. Sensitivity analyses to evaluate effects of soil property uncertainties should be conducted and considered in developing the design response spectrum.

3.4.2 Deterministic maximum considered earthquake. It is required that ground motions for the deterministic maximum considered earthquake be based on characteristic earthquakes on all known active faults in a region. As defined in Sec. 3.1.3, the magnitude of a characteristic earthquake on a given fault should be a best-estimate of the maximum magnitude capable for that fault but not less than the largest magnitude that has occurred historically on the fault. The maximum magnitude should be estimated considering all seismic-geologic evidence for the fault, including fault length and paleoseismic observations. For faults characterized as having more than a single segment, the potential for rupture of multiple segments in a single earthquake should be considered in assessing the characteristic maximum magnitude for the fault.

3.5 SITE CLASSIFICATION FOR SEISMIC DESIGN

3.5.1 Site Class Definitions. Based on the studies and observations discussed in Sec. 3.3-2, the site categories in the 2003 Provisions are defined in terms of the small-strain shear wave velocity in the top 100 ft (30 m) of the profile, $\overline{v}_s$, as might be inferred from travel time for a shear wave to travel from the surface to a depth of 100 ft (30m). If shear wave velocities are available for the site, they should be used to classify the site.

However, in recognition of the fact that in many cases the shear wave velocities are not available, alternative definitions of the site classes also are included in the 2003 Provisions. They use the standard penetration resistance for cohesionless and cohesive soils and rock and the undrained shear strength for cohesive soils only. These alternative definitions are rather conservative since the correlation between site amplification and these geotechnical parameters is more uncertain than the correlation with $\overline{v}_s$. That is, there will be cases where the values of $F_a$ and $F_v$ will be smaller if the site
category is based on \( \bar{v} \), rather than on the geotechnical parameters. Also, the site category definitions should not be interpreted as implying any specific numerical correlation between shear-wave velocity and standard penetration resistance or shear strength.

Equation 3.5-1 is for inferring the average shear-wave velocity to a depth of 100 ft (30m) at a site. Equation 3.5-1 specifies that the average velocity is given by the sum of the thicknesses of the geologic layers in the upper 100 ft divided by the sum of the times for a shear wave to travel through each layer, where travel time for each layer is specified by the ratio of the thickness and the shear wave velocity for the layer. It is important that this method of averaging be used as it may result in a significantly lower effective average shear wave velocity than the velocity that would be obtained by averaging the velocities of the individual layers directly.

Equation 3.5-2 is for classifying the site using the standard penetration resistance (\( N \)-value) for cohesionless soils, cohesive soils, and rock in the upper 100 ft (30 m). A method of averaging analogous to the method of Equation 3.5-1 for shear wave velocity is used. The maximum value of \( N \) that can be used for any depth of measurement in soil or rock is 100 blows/ft.

Equations 3.5-3 and 3.5-4 are for classifying the site using the standard penetration resistance of cohesionless soil layers, \( N_{ch} \), and the undrained shear strength of cohesive soil layers, \( s_u \), within the top 100 ft (30 m). These equations are provided as an alternative to using Eq. 3.5-2 for which \( N \)-values in all geologic materials in the top 100 ft (30 m) are used. When using Eq. 3.5-3 and 3.5-4, only the thicknesses of cohesionless soils and cohesive soils within the top 100 ft (30 m) are used.

As indicated in Sec. 3.3-2 and 3.5-1, soils classified as Site Class F according to the definitions in Sec. 3.5-1 require site-specific evaluations. An exception is made, however, for liquefiable sites where the structure has a fundamental period of vibration equal to or less than 0.5 second. For such structures, values of \( F_a \) and \( F_v \) for the site may be determined using the site class definitions and criteria in Sec. 3.5-1 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 generally attenuated due to liquefaction whereas long-period ground motions may be amplified. This exception is only for the purposes of defining the site class and obtaining site coefficients. It is still required to assess liquefaction potential and its effects on structures as a ground failure hazard as specified in Chapter 7.

### 3.5.2 Steps for classifying a site

A step-by-step procedure for classifying a site is given in the Provisions. Although the procedure and criteria in Sec. 3.5.1 and 3.5.2 are straightforward, there are aspects of these assessments that may require additional judgment and interpretation. Highly variable subsurface conditions beneath a building footprint could result in overly conservative or unconservative site classification. Isolated soft soil layers within an otherwise firm soil site may not affect the overall site response if the predominant soil conditions do not include such strata. Conversely, site response studies have shown that continuous, thin, soft clay strata may increase the site amplification.

The site class should reflect the soil conditions that will affect the ground motion input to the structure or a significant portion of the structure. For structures receiving substantial ground motion input from shallow soils (e.g. structures with shallow spread footings, laterally flexible piles, or structures with basements where it is judged that substantial ground motion input to the structure may come through the side walls), it is reasonable to classify the site on the basis of the top 100 ft (30 m) of soils below the ground surface. Conversely, for structures with basements supported on firm soils or rock below soft soils, it is reasonable to classify the site on the basis of the soils or rock below the mat, if it can be justified that the soft soils contribute very little to the response of the structure.

Buildings on sloping bedrock sites and/or having highly variable soil deposits across the building area require careful study since the input motion may vary across the building (for example, if a portion of the building is on rock and the rest is over weak soils). Site-specific studies including two- or three-
dimensional modeling may be appropriate in such cases to evaluate the subsurface conditions and site
and superstructure response. Other conditions that may warrant site-specific evaluation include the
presence of low shear wave velocity soils below a depth of 100 ft (30 m), location of the site near the
edge of a filled-in basin, or other subsurface or topographic conditions with strong two- and three-
dimensional site-response effects. Individuals with appropriate expertise in seismic ground motions
should participate in evaluations of the need for and nature of such site-specific studies.

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