2. A jurisdiction with low variation in ground motion across the area might stipulate particular values of ground motion rather than requiring the use of maps.
3. An area with unusual soils might require use of a particular site class unless a geotechnical investigation proves a better site class.

C11.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

The 2002 edition of the standard included a new provision of minimum lateral force for Seismic Design Category A structures. The minimum load is a structural integrity issue related to the load path. It is intended to specify design forces in excess of wind loads in heavy low-rise construction. The design calculation in Section 1.4.2 of the standard is simple and easily done to ascertain if the seismic load or the wind load governs. This provision requires a minimum lateral force of 1% of the total gravity load assigned to a story to ensure general structural integrity.

Seismic Design Category A is assigned when the MCE ground motions are below those normally associated with hazardous damage. Damaging earthquakes are not unknown or impossible in such regions, however, and ground motions close to such events may be large enough to produce serious damage. Providing a minimum level of resistance reduces both the radius over which the ground motion exceeds structural capacities and resulting damage in such rare events. There are reasons beyond seismic risk for minimum levels of structural integrity.

The requirements for SDC A in Section 1.4 are all minimum strengths for structural elements stated as forces at the level appropriate for direct use in the strength design load combinations of Section. 2.3. The two fundamental requirements are a minimum strength for a structural system to resist lateral forces (Section 1.4.2) and a minimum strength for connections of structural members (Section 1.4.3).

For many buildings, the wind force controls the strength of the lateral-force-resisting system, but for low-rise buildings of heavy construction with large plan aspect ratios, the minimum lateral force specified in Section 1.4.2 may control. Note that the requirement is for strength and not for toughness, energy-dissipation capacity, or some measure of ductility. The force level is not tied to any postulated seismic ground motion. The boundary between SDCs A and B is based on a spectral response acceleration of 25% of gravity (MCE level) for short-period structures; clearly the 1% acceleration level (from Eq. (1.4-1)) is far smaller. For ground motions below the A/B boundary, the spectral displacements generally are on the order of a few inches or less depending on period. Experience has shown that even a minimal strength is beneficial in providing resistance to small ground motions, and it is an easy provision to implement in design. The low probability of motions greater than the MCE is a factor in taking the simple approach without requiring details that would produce a ductile response. Another factor is that larger design forces are specified in Section 1.4.3 for connections between main elements of the lateral force load path.

The minimum connection force is specified in three ways: a general minimum horizontal capacity for all connections; a special minimum for horizontal restraint of in-line beams and trusses, which also includes the live load on the member; and a special minimum for horizontal restraint of concrete and masonry walls perpendicular to their plane (Section 1.4.4). The 5% coefficient used for the first two is a simple and convenient value that provides some margin over the minimum strength of the system as a whole.

C11.8 GEOLOGIC HAZARDS AND GEOTECHNICAL INVESTIGATION

In addition to this commentary, Part 3 of the 2009 NEHRP recommended provisions (FEMA 2009) includes additional and more detailed discussion and guidance on evaluation of geologic hazards and determination of seismic lateral pressures.

C11.8.1 Site Limitation for Seismic Design Categories E and F. Because of the difficulty of designing a structure for the direct shearing displacement of fault rupture and the relatively high seismic activity of SDCs E and F, locating a structure on an active fault that has the potential to cause rupture of the ground surface at the structure is prohibited.

C11.8.2 Geotechnical Investigation Report Requirements for Seismic Design Categories C through F. Earthquake motion is only one factor in assessing potential for geologic and seismic hazards. All of the listed hazards can lead to surface ground displacements with potential adverse consequences to structures. Finally, hazard identification alone has little value unless mitigation options are also identified.

C11.8.3 Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F. New provisions for computing peak ground acceleration for soil liquefaction and stability evaluations have been introduced in this section. Of particular note in this section is the explicitly stated requirement that liquefaction must now be evaluated for the MCE_G ground motion. These provisions include maps of the maximum considered earthquake geometric mean (MCE_G) peak ground acceleration (PGA) for Site Class B bedrock plus a site-coefficient (F_{PGA}) table to convert the PGA value to one adjusted for site class effects (PGA_M).

A requirement, similar to the one in Section 11.4.4, has been added to the provisions to take the larger of the F_{PGA} for Site Classes C and D to conservatively account for the amplification when the site is known to consist of soil that is not in Site Class E or F.

The equation used to derive the F_{PGA} values is similar to Eqs. (C11.4-1) and (C11.4-2) for F_a and F_v; it is as follows:

$$F_{PGA} = \exp\left[-0.604 \ln\left(\frac{\overline{v}_s}{760}\right) - 0.150 \begin{bmatrix} \exp\{-0.00701(\min(\overline{v}_s, 760) - 360)\} - \\ \exp\{-0.00701 \times 400\} \end{bmatrix} \times \ln\left(\frac{PGA + 0.1}{0.1}\right)\right] \quad \text{(C11.8-1)}$$

In Eq. (C11.8-1), $\overline{v}_s$ is in units of m/s and PGA is in units of g. Velocities measured in ft/s can be converted to m/s by multiplying by 0.3048. To obtain the F_{PGA} for $\overline{v}_s < 180$ m/s ($\overline{v}_s < 590$ ft/s), the +1/2 standard-deviation correction described for Site Class E in Section C11.4.4 would need to be applied to the natural logarithm of F_{PGA}. The standard deviation is 0.70.

PGA Provisions. Item 2 of Section 11.8.3 states that peak ground acceleration shall be determined based on either a site-specific study, taking into account soil amplification effects, or using Eq. (11.8-1), for which MCE_G peak ground acceleration is obtained from national maps of PGA for bedrock Site Class B multiplied by a site coefficient (F_{PGA}) to obtain peak ground acceleration for other site classes (PGA_M). This methodology for determining peak ground acceleration for liquefaction evaluations improved the methodology in ASCE 7-05 by using mapped

PGA rather than the approximation for PGA by the ratio $S_s/2.5$. Furthermore, in the central and eastern United States, the ratio $S_s/2.5$ tends to underestimate PGA. $S_s/2.5$ is applicable for bedrock Site Class B and thus could be used as input at depth to a site response analysis under the provisions of ASCE 7-05. The use of Eq. (11.8-1) provides an alternative to conducting site response analysis using rock PGA by providing a site-adjusted ground surface acceleration (PGA_M) that can directly be applied in the widely used empirical correlations for assessing liquefaction potential. Correlations for evaluating liquefaction potential are elaborated on in Resource Paper RP 12, "Evaluation of Geologic Hazards and Determination of Seismic Lateral Earth Pressures," published in the 2009 NEHRP provisions (FEMA 2009).

Maps of MCE_G PGA for Site Class B bedrock, similar to maps of S_s and S_1, are shown in Figs. 22-9 to 22-13 in Chapter 22. Similar to adjustments for the bedrock spectral response accelerations for site response through the F_a and F_v coefficients, bedrock motions for PGA are adjusted for these same site effects using a site coefficient, F_{PGA}, that depends on the level of ground shaking in terms of PGA and the stiffness of the soil, typically defined in terms of the shear-wave velocity in the upper 30 m (98.4 ft) of geologic profile, V_{s30}. Values of F_{PGA} are presented in Table 11.8-1, and the adjustment is made through Eq. (11.8-1), i.e., $PGA_M = F_{PGA}$ PGA, where PGA_M is peak ground acceleration adjusted for site class. The method of determining site class, used in the determination of F_a and F_v, is also identical to that in the present and previous ASCE 7 documents.

There is an important difference in the derivation of the PGA maps and the maps of S_s and S_1 in ASCE 7-10. Unlike previous editions of ASCE 7, the S_s and S_1 maps in ASCE 7-10 were derived for the "maximum direction shaking" and are risk based rather than hazard based. However, the PGA maps have been derived based on the geometric mean of the two horizontal components of motion. The geometric mean was used in the PGA maps rather than the PGA for the maximum direction shaking to ensure that there is consistency between the determination of PGA and the basis of the simplified empirical field procedure for estimating liquefaction potential based on results of standard penetration tests (SPTs), cone penetrometer tests (CPTs), and other similar field investigative methods. When these correlations were originally derived, the geomean (or a similar metric) of peak ground acceleration at the ground surface was used to identify the cyclic stress ratio for sites with or without liquefaction. The resulting envelopes of data define the liquefaction cyclic resistance ratio (CRR). Rather than reevaluating these case histories for the "maximum direction shaking," it was decided to develop maps of the geomean PGA and to continue using the existing empirical methods.

Liquefaction Evaluation Requirements. Beginning with ASCE 7-02, it has been the intent that liquefaction potential be evaluated at MCE ground motion levels. There was ambiguity in the previous requirement in ASCE 7-05 as to whether liquefaction potential should be evaluated for the MCE or for the design earthquake. Paragraph 2 of Section 11.8.3 of ASCE 7-05 stated that liquefaction potential would be evaluated for the design earthquake; it also stated that in the absence of a site-specific study, peak ground acceleration shall be assumed to be equal to $S_s/2.5$ (S_s is the MCE short-period response spectral acceleration on Site Class B rock). There has also been a difference in provisions between ASCE 7-05 and the 2006 edition of the IBC, in which Section 1802.2.7 stated that liquefaction shall be evaluated for the design earthquake ground motions and the default value of peak ground acceleration in the absence of a site-specific study was given as $S_{DS}/2.5$ (S_{DS} is the short-period site-adjusted design response spectral acceleration). ASCE 7-10, in item 2 of Section 11.8.3 and Eq. (11.8-1), requires explicitly that liquefaction potential be evaluated based on the MCE_G peak ground acceleration.

The explicit requirement in ASCE 7-10 to evaluate liquefaction for MCE ground motion rather than to design earthquake ground motion ensures that the full potential for liquefaction is addressed during the evaluation of structure stability, rather than a lesser level when the design earthquake is used. This change also ensures that, for the MCE ground motion, the performance of the structure is considered under a consistent hazard level for the effects of liquefaction, such as collapse prevention or life safety, depending on the risk category for the structure (Fig. C11.5-1). By evaluating liquefaction for the MCE rather than the design earthquake peak ground acceleration, the ground motion for the liquefaction assessment increases by a factor of 1.5. This increase in peak ground acceleration to the MCE level means that sites that previously were nonliquefiable could now be liquefiable, and sites where liquefaction occurred to a limited extent under the design earthquake could undergo more liquefaction, in terms of depth and lateral extent. Some mechanisms that are directly related to the development of liquefaction, such as lateral spreading and flow or ground settlement, could also increase in severity.

This change in peak ground acceleration level for the liquefaction evaluation addressed an issue that has existed and has periodically been discussed since the design earthquake concept was first suggested in the 1990s. The design earthquake ground motion was obtained by multiplying the MCE ground motion by a factor of $2/3$ to account for a margin in capacity in most buildings. Various calibration studies at the time of code development concluded that for the design earthquake, most buildings had a reserve capacity of more than 1.5 relative to collapse. This reserve capacity allowed the spectral accelerations for the MCE to be reduced using a factor of $2/3$, while still achieving safety from collapse. However, liquefaction potential is evaluated at the selected MCE_G peak ground acceleration and is typically determined to be acceptable if the factor of safety is greater than 1.0, meaning that there is no implicit safety margin on liquefaction potential. By multiplying peak ground acceleration by a factor of $2/3$, liquefaction would be assessed at an effective return period or probability of exceedance different than that for the MCE. However, ASCE 7-10 requires that liquefaction be evaluated for the MCE.

Item 3 of Section 11.8.3 of the ASCE 7-10 standard lists the various potential consequences of liquefaction that must be assessed; soil downdrag and loss in lateral soil reaction for pile foundations are additional consequences that have been included in this paragraph. This section of the new provisions, as in previous editions, does not present specific seismic criteria for the design of the foundation or substructure, but item 4 does state that the geotechnical report must include discussion of possible measures to mitigate these consequences.

A liquefaction resource document has been prepared in support of these revisions to Section 11.8.3. The resource document "Evaluation of Geologic Hazards and Determination of Seismic Lateral Earth Pressures," includes a summary of methods that are currently being used to evaluate liquefaction potential and the limitations of these methods. This summary appears as Resource Paper RP 12 in the 2009 NEHRP provisions (FEMA 2009). The resource document summarizes alternatives for evaluating liquefaction potential, methods for evaluating the possible consequences of liquefaction (e.g., loss of ground support and increased lateral earth pressures) and methods of mitigating the liquefaction hazard. The resource document also identifies alternate methods of evaluating liquefaction hazards, such as analytical and physical modeling. Reference is made to the use of nonlinear effective stress methods for modeling the buildup in pore water pressure during seismic events at liquefiable sites.

Evaluation of Dynamic Seismic Lateral Earth Pressures. The dynamic lateral earth pressure on basement and retaining walls during earthquake ground shaking is considered to be an earthquake load, E, for use in design load combinations. This dynamic earth pressure is superimposed on the preexisting static lateral earth pressure during ground shaking. The preexisting static lateral earth pressure is considered to be an H load.

C11.9 VERTICAL GROUND MOTIONS FOR SEISMIC DESIGN

C11.9.2 MCE$_R$ Vertical Response Spectrum. Previous editions of ASCE 7 do not provide adequate guidance regarding procedures for estimating vertical ground motion levels for use in earthquake-resistant design. Historically, the amplitude of vertical ground motion has been inferred to be two-thirds ($2/3$) the amplitude of the horizontal ground motion. However, studies of horizontal and vertical ground motions over the past 25 years have shown that such a simple approach is not valid in many situations (e.g., Bozorgnia and Campbell 2004, and references therein) for the following main reasons: (1) vertical ground motion has a larger proportion of short-period (high-frequency) spectral content than horizontal ground motion, and this difference increases with decreasing soil stiffness, and (2) vertical ground motion attenuates at a higher rate than horizontal ground motion, and this difference increases with decreasing distance from the earthquake. The observed differences in the spectral content and attenuation rate of vertical and horizontal ground motion lead to the following observations regarding the vertical/horizontal (V/H) spectral ratio (Bozorgnia and Campbell 2004):

1. The V/H spectral ratio is sensitive to spectral period, distance from the earthquake, local site conditions, and earthquake magnitude and is insensitive to earthquake mechanism and sediment depth;
2. The V/H spectral ratio has a distinct peak at short periods that generally exceeds $2/3$ in the near-source region of an earthquake; and
3. The V/H spectral ratio is generally less than $2/3$ at mid-to-long periods.

Therefore, depending on the period, the distance to the fault, and the local site conditions of interest, use of the traditional $2/3$ V/H spectral ratio can result in either an under- or overestimation of the expected vertical ground motions.

The procedure for defining the MCE$_R$ vertical response spectrum in ASCE 7 is a modified version of the procedure taken from the 2009 *NEHRP Provisions*. Unlike the procedure contained in the 2009 *NEHRP Provisions*, the procedure provided in Section 11.9 is keyed to the MCE$_R$ spectral response acceleration parameter at short periods, S_{MS}. The procedure is based on the studies of horizontal and vertical ground motions conducted by Campbell and Bozorgnia (2003) and Bozorgnia and Campbell (2004). These procedures are also generally compatible with the general observations of Abrahamson and Silva (1997) and Silva (1997) and the proposed design procedures of Elnashai (1997). The procedure has been modified to express the vertical ground motions in terms of MCE$_R$ ground motions instead of design ground motions.

To be consistent with the shape of the horizontal design response spectrum, the vertical design response spectrum has four regions defined by the vertical period of vibration (T_v). Based on the study of Bozorgnia and Campbell (2004), the periods that define these regions are approximately constant with respect to the magnitude of the earthquake, the distance from the earthquake, and the local site conditions. In this respect, the shape of the vertical response spectrum is simpler than that of the horizontal response spectrum.

The equations that are used to define the design vertical response spectrum are based on three observations made by Bozorgnia and Campbell (2004):

1. The short-period part of the 5% damped vertical response spectrum is controlled by the spectral acceleration at $T_v = 0.1$ s;
2. The mid-period part of the vertical response spectrum is controlled by a spectral acceleration that decays as the inverse of the 0.75 power of the vertical period of vibration ($T_v^{-0.75}$); and
3. The short-period part of the V/H spectral ratio is a function of the local site conditions, the distance from the earthquake (for sites located within about 30 mi (60 km) of the fault), and the earthquake magnitude (for soft sites).

ASCE 7 does not include seismic design maps for the vertical spectral acceleration at $T_v = 0.1$ s and does not preserve any information on the earthquake magnitudes or the source-to-site distances that contribute to the horizontal spectral accelerations that are mapped. Therefore, the general procedure recommended by Bozorgnia and Campbell (2004) was modified to use only those horizontal spectral accelerations that are available from the seismic design maps, as follows:

1. Estimate the vertical spectral acceleration at $T_v = 0.1$ s from the ratio of this spectral acceleration to the horizontal spectral acceleration at $T = 0.2$ s for the Site Class B/C boundary (i.e., the boundary between Site Classes B and C $\bar{v}_s = 2{,}500$ ft/s ($\bar{v}_s = 760$ m/s), the reference site condition for the 2008 U.S. Geological Survey National Seismic Hazard Maps). For earthquakes and distances for which the vertical spectrum might be of engineering interest (magnitudes greater than 6.5 and distances less than 30 mi (60 km), this ratio is approximately 0.8 for all site conditions (Campbell and Bozorgnia 2003).
2. Estimate the horizontal spectral acceleration at $T = 0.2$ s from the Next Generation Attenuation (NGA) relationship of Campbell and Bozorgnia (2008) for magnitudes greater than 6.5 and distances ranging between 1 and 30 mi (1 and 60 km) for the Site Class B/C boundary $\bar{v}_s = 2{,}500$ ft/s ($\bar{v}_s = 760$ m/s). The relationship of Campbell and Bozorgnia (2008), rather than that of Campbell and Bozorgnia (2003), was used for this purpose to be consistent with the development of the 2008 U.S. Geological Survey National Seismic Hazard Maps, which use the NGA attenuation relationships to estimate horizontal ground motions in the western United States. Similar results were found for the other two NGA relationships that were used to develop the seismic hazard and design maps (Boore and Atkinson 2008; Chiou and Youngs 2008).
3. Use the dependence between the horizontal spectral acceleration at $T = 0.2$ s and source-site distance estimated in Item 2 and the relationship between the V/H spectral ratio, source-site distance, and local site conditions in Bozorgnia and Campbell (2004) to derive a relationship between the vertical spectral acceleration and the mapped MCE$_R$ spectral response acceleration parameter at short periods, S_S.
4. Use the dependence between the vertical spectral acceleration and the mapped MCE$_R$ spectral response acceleration

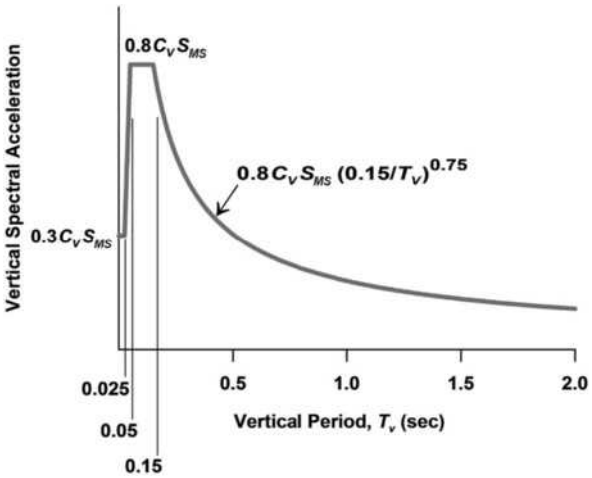

FIGURE C11.9-1 Illustrative Example of the Vertical Response Spectrum

parameter at short periods, S_S, in Item 3 to derive a vertical coefficient, C_v, that when multiplied by 0.8 and the MCE_R horizontal response acceleration at short periods, S_{MS}, results in an estimate of the design vertical spectral acceleration at $T_v = 0.1$ s.

The following description of the detailed procedure listed in Section 11.9.2 refers to the illustrated MCE_R vertical response spectrum in Fig. C11.9-1.

Vertical Periods Less Than or Equal to 0.025 s. Eq. (11.9-1) defines that part of the MCE_R vertical response spectrum that is controlled by the vertical peak ground acceleration. The 0.3 factor was approximated by dividing the 0.8 factor that represents the ratio between the vertical spectral acceleration at $T_v = 0.1$ s and the horizontal spectral acceleration at $T = 0.2$ s by 2.5, the factor that represents the ratio between the MCE_R horizontal spectral acceleration at $T = 0.2$ s, S_{MS}, and the zero-period acceleration used in the development of the MCE_R horizontal response spectrum. The vertical coefficient, C_v, in Table 11.9-1 accounts for the dependence of the vertical spectral acceleration on the amplitude of the horizontal spectral acceleration and the site dependence of the V/H spectral ratio as determined in Items 3 and 4 above. The factors are applied to S_{MS} rather than to S_S because S_{MS} already includes the effects of local site conditions.

Vertical Periods Greater Than 0.025 s and Less Than or Equal to 0.05 s. Eq. (11.9-2) defines that part of the MCE_R vertical response spectrum that represents the linear transition from the part of the spectrum that is controlled by the vertical peak ground acceleration and the part of the spectrum that is controlled by the dynamically amplified short-period spectral plateau. The factor of 20 is the factor that is required to make this transition continuous and piecewise linear between these two adjacent parts of the spectrum.

Vertical Periods Greater Than 0.05 s and Less Than or Equal to 0.15 s. Eq. (11.9-3) defines that part of the MCE_R vertical response spectrum that represents the dynamically amplified short-period spectral plateau.

Vertical Periods Greater Than 0.15 s and Less Than or Equal to 2.0 s. Eq. (11.9-4) defines that part of the MCE_R vertical response spectrum that decays with the inverse of the vertical period of vibration raised to the 0.75 power.

Two limits are imposed on the MCE_R vertical response spectrum defined by Eqs. (11.9-1) through (11.9-4) and illustrated in Fig. C11.9-1. The first limit restricts the applicability of the vertical response spectrum to a maximum vertical period of vibration of 2 s. This limit accounts for the fact that such large vertical periods are rare (structures are inherently stiff in the vertical direction) and that the vertical spectrum might decay differently with period at longer periods. There is an allowance for developing a site-specific MCE_R vertical response spectrum if this limit is exceeded (see Section 11.4 or Chapter 21 for guidance on applying site-specific methods). The second limit restricts the MCE_R vertical response spectrum to be no less than 50% of the MCE_R horizontal response spectrum. This limit accounts for the fact that a V/H spectral ratio of one-half (1/2) is a reasonable, but somewhat conservative, lower bound over the period range of interest, based on the results of Campbell and Bozorgnia (2003) and Bozorgnia and Campbell (2004).

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CHAPTER C12
SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

C12.1 STRUCTURAL DESIGN BASIS

The performance expectations for structures designed in accordance with this standard are described in Sections C11.1 and C11.5. Structures designed in accordance with the standard are likely to have a low probability of collapse but may suffer serious structural damage if subjected to the risk-targeted maximum considered earthquake (MCE_R) or stronger ground motion.

Although the seismic requirements of the standard are stated in terms of forces and loads, there are no external forces applied to the structure during an earthquake as, for example, is the case during a windstorm. The design forces are intended only as approximations to generate internal forces suitable for proportioning the strength and stiffness of structural elements and for estimating the deformations (when multiplied by the deflection amplification factor, C_d) that would occur in the same structure in the event of design earthquake (not MCE_R) ground motion.

C12.1.1 Basic Requirements. Chapter 12 of the standard sets forth a set of coordinated requirements that must be used together. The basic steps in structural design of a building structure for acceptable seismic performance are as follows:

1. Select gravity- and seismic force-resisting systems appropriate to the anticipated intensity of ground shaking. Section 12.2 sets forth limitations depending on the Seismic Design Category.
2. Configure these systems to produce a continuous, regular, and redundant load path so that the structure acts as an integral unit in responding to ground shaking. Section 12.3 addresses configuration and redundancy issues.
3. Analyze a mathematical model of the structure subjected to lateral seismic motions and gravity forces. Sections 12.6 and 12.7 set forth requirements for the method of analysis and for construction of the mathematical model. Sections 12.5, 12.8, and 12.9 set forth requirements for conducting a structural analysis to obtain internal forces and displacements.
4. Proportion members and connections to have adequate lateral and vertical strength and stiffness. Section 12.4 specifies how the effects of gravity and seismic loads are to be combined to establish required strengths, and Section 12.12 specifies deformation limits for the structure.

One- to three-story structures with shear wall or braced frame systems of simple configuration may be eligible for design under the simplified alternative procedure contained in Section 12.14. Any other deviations from the requirements of Chapter 12 are subject to approval by the authority having jurisdiction (AHJ) and must be rigorously justified, as specified in Section 11.1.4.

The baseline seismic forces used for proportioning structural elements (individual members, connections, and supports) are static horizontal forces derived from an elastic response spectrum procedure. A basic requirement is that horizontal motion can come from any direction relative to the structure, with detailed requirements for evaluating the response of the structure provided in Section 12.5. For most structures, the effect of vertical ground motions is not analyzed explicitly; it is implicitly included by adjusting the load factors (up and down) for permanent dead loads, as specified in Section 12.4. Certain conditions requiring more detailed analysis of vertical response are defined in Chapters 13 and 15 for nonstructural components and nonbuilding structures, respectively.

The basic seismic analysis procedure uses response spectra that are representative of, but substantially reduced from, the anticipated ground motions. As a result, at the MCE_R level of ground shaking, structural elements are expected to yield, buckle, or otherwise behave inelastically. This approach has substantial historical precedent. In past earthquakes, structures with appropriately ductile, regular, and continuous systems that were designed using *reduced* design forces have performed acceptably. In the standard, such design forces are computed by dividing the forces that would be generated in a structure behaving elastically when subjected to the design earthquake ground motion by the response modification coefficient, R, and this design ground motion is taken as two-thirds of the MCE_R ground motion.

The intent of R is to reduce the demand determined, assuming that the structure remains elastic at the design earthquake, to target the development of the first significant yield. This reduction accounts for the displacement ductility demand, R_d, required by the system and the inherent overstrength, Ω, of the seismic force-resisting system (SFRS) (Fig. C12.1-1). Significant yield is the point where complete plastification of a critical region of the SFRS first occurs (e.g., formation of the first plastic hinge in a moment frame), and the stiffness of the SFRS to further increases in lateral forces decreases as continued inelastic behavior spreads within the SFRS. This approach is consistent with member-level ultimate strength design practices. As such, first significant yield should not be misinterpreted as the point where first yield occurs in any member (e.g., 0.7 times the yield moment of a steel beam or either initial cracking or initiation of yielding in a reinforcing bar in a reinforced concrete beam or wall).

Fig. C12.1-1 shows the lateral force versus deformation relation for an archetypal moment frame used as an SFRS. First significant yield is shown as the lowest plastic hinge on the force–deformation diagram. Because of particular design rules and limits, including material strengths in excess of nominal or project-specific design requirements, structural elements are stronger by some degree than the strength required by analysis. The SFRS is therefore expected to reach first significant yield for forces in excess of design forces. With increased lateral loading,

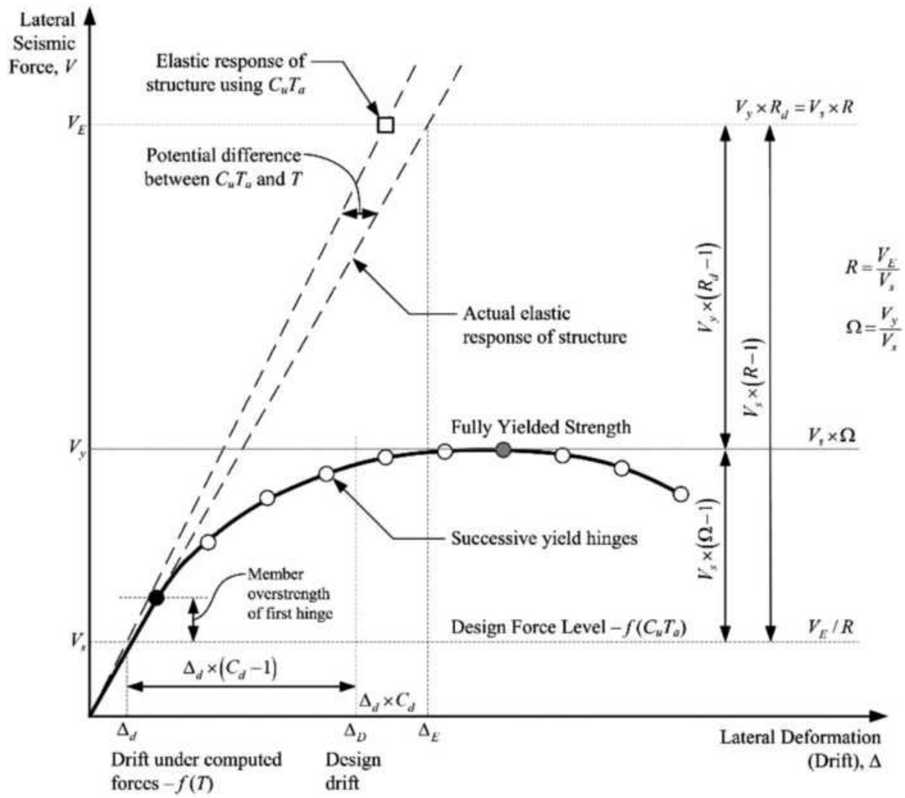


FIGURE C12.1-1 Inelastic Force–Deformation Curve

additional plastic hinges form and the resistance increases at a reduced rate (following the solid curve) until the maximum strength is reached, representing a fully yielded system. The maximum strength developed along the curve is substantially higher than that at first significant yield, and this margin is referred to as the system overstrength capacity. The ratio of these strengths is denoted as Ω. Furthermore, the figure illustrates the potential variation that can exist between the actual elastic response of a system and that considered using the limits on the fundamental period (assuming 100% mass participation in the fundamental mode—see Section C12.8.6). Although not a concern for strength design, this variation can have an effect on the expected drifts.

The system overstrength described above is the direct result of overstrength of the elements that form the SFRS and, to a lesser extent, the lateral force distribution used to evaluate the inelastic force–deformation curve. These two effects interact with applied gravity loads to produce sequential plastic hinges, as illustrated in the figure. This member overstrength is the consequence of several sources. First, material overstrength (i.e., actual material strengths higher than the nominal material strengths specified in the design) may increase the member overstrength significantly. For example, a recent survey shows that the mean yield strength of ASTM A36 steel is about 30% to 40% higher than the specified yield strength used in design calculations. Second, member design strengths usually incorporate a strength reduction or resistance factor, ϕ, to produce a low probability of failure under design loading. It is common to not include this factor in the member load-deformation relation when evaluating the seismic response of a structure in a nonlinear structural analysis. Third, designers can introduce additional strength by selecting sections or specifying reinforcing patterns that exceed those required by the computations. Similar situations occur where prescriptive minimums of the standard, or of the referenced design standards, control the design. Finally, the design of many flexible structural systems (e.g., moment-resisting frames) can be controlled by the drift rather than strength, with sections selected to control lateral deformations rather than to provide the specified strength.

The result is that structures typically have a much higher lateral strength than that specified as the minimum by the standard, and the first significant yielding of structures may occur at lateral load levels that are 30% to 100% higher than the prescribed design seismic forces. If provided with adequate ductile detailing, redundancy, and regularity, full yielding of structures may occur at load levels that are two to four times the prescribed design force levels.

Most structural systems have some elements whose action cannot provide reliable inelastic response or energy dissipation. Similarly, some elements are required to remain essentially elastic to maintain the structural integrity of the structure (e.g., columns supporting a discontinuous SFRS). Such elements and actions must be protected from undesirable behavior by considering that the actual forces within the structure can be significantly larger than those at first significant yield. The standard specifies an overstrength factor, Ω_0, to amplify the prescribed seismic forces for use in design of such elements and for such actions. This approach is a simplification to determining the maximum forces that could be developed in a system and the distribution of these forces within the structure. Thus, this specified overstrength factor is neither an upper nor a lower bound; it is simply an approximation specified to provide a nominal degree of protection against undesirable behavior.

The elastic deformations calculated under these reduced forces (*see* Section C12.8.6) are multiplied by the deflection amplification factor, C_d, to estimate the deformations likely to result from the design earthquake ground motion. This factor was first introduced in ATC 3-06 (ATC 1978). For a vast majority of systems, C_d is less than R, with a few notable exceptions, where inelastic drift is strongly coupled with an increased risk of collapse (e.g., reinforced concrete bearing walls). Research over the past 30 years has illustrated that inelastic displacements may be significantly greater than Δ_E for many structures and less than Δ_E for others. Where C_d is substantially less than R, the system is considered to have damping greater than the nominal 5% of critical damping. As set forth in Section 12.12 and Chapter 13, the amplified deformations are used to assess story drifts and to determine seismic demands on elements of the structure that are not part of the seismic force-resisting system and on nonstructural components within structures.

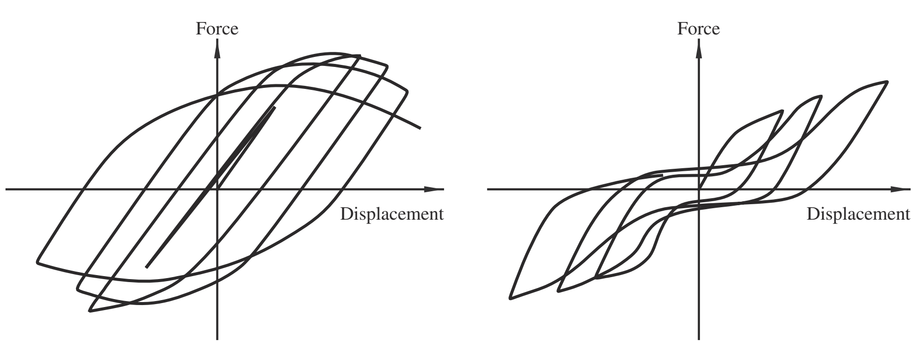
Fig. C12.1-1 illustrates the significance of seismic design parameters contained in the standard, including the response modification coefficient, R; the deflection amplification factor, C_d; and the overstrength factor, Ω_0. The values of these parameters, provided in Table 12.2-1, as well as the criteria for story drift and P-delta effects, have been established considering the characteristics of typical properly designed structures. The provisions of the standard anticipate an SFRS with redundant characteristics wherein significant system strength above the level of first significant yield can be obtained by plastification at other critical locations in the structure before the formation of a collapse mechanism. If excessive "optimization" of a structural design is performed with lateral resistance provided by only a few elements, the successive yield hinge behavior depicted in Fig. C12.1-1 is not able to form, the actual overstrength (Ω) is small, and use of the seismic design parameters in the standard may not provide the intended seismic performance.

The response modification coefficient, R, represents the ratio of the forces that would develop under the specified ground motion if the structure had an entirely linear-elastic response to the prescribed design forces (Fig. C12.1-1). The structure must be designed so that the level of significant yield exceeds the prescribed design force. The ratio R_d, expressed as $R_d = V_E/V_S$, where V_E is the elastic seismic force demand and V_S is the prescribed seismic force demand, is always larger than 1.0; thus, all structures are designed for forces smaller than those the design ground motion would produce in a structure with a completely linear-elastic response. This reduction is possible for a number of reasons. As the structure begins to yield and deform inelastically, the effective period of response of the structure lengthens, which results in a reduction in strength demand for most structures. Furthermore, the inelastic action results in a significant amount of energy dissipation (hysteretic damping) in addition to other sources of damping present below significant yield. The combined effect, which is known as the ductility reduction, explains why a properly designed structure with a fully yielded strength (V_y in Fig. C12.1-1) that is significantly lower than V_E can be capable of providing satisfactory performance under the design ground motion excitations.

The energy dissipation resulting from hysteretic behavior can be measured as the area enclosed by the force–deformation curve of the structure as it experiences several cycles of excitation. Some structures have far more energy dissipation capacity than others. The extent of energy dissipation capacity available depends largely on the amount of stiffness and strength degradation the structure undergoes as it experiences repeated cycles of inelastic deformation. Fig. C12.1-2 shows representative load deformation curves for two simple substructures, such as a beam–column assembly in a frame. Hysteretic curve (a) in the figure represents the behavior of substructures that have been detailed for ductile behavior. The substructure can maintain almost all of its strength and stiffness over several large cycles of inelastic deformation. The resulting force–deformation "loops" are quite wide and open, resulting in a large amount of energy dissipation. Hysteretic curve (b) represents the behavior of a substructure that has much less energy dissipation than that for the substructure (a) but has a greater change in response period. The structural response is determined by a combination of energy dissipation and period modification.

The principles of this section outline the conceptual intent behind the seismic design parameters used by the standard. However, these parameters are based largely on engineering judgment of the various materials and performance of structural systems in past earthquakes and cannot be directly computed using the relationships presented in Fig. C12.1-1. The seismic design parameters chosen for a specific project or system should be chosen with care. For example, lower values should be used for structures possessing a low degree of redundancy wherein all the plastic hinges required for the formation of a mechanism may be formed essentially simultaneously and at a force level close to the specified design strength. This situation can result in considerably more detrimental P-delta effects. Because it is difficult for individual designers to judge the extent to which the value of R should be adjusted based on the inherent redundancy of their designs, Section 12.3.4 provides the redundancy factor, ρ, that is typically determined by being based on the removal of individual seismic force-resisting elements.

Higher order seismic analyses are permitted for any structure and are required for some structures (*see* Section 12.6); lower limits based on the equivalent lateral force procedure may, however, still apply.



(a) Ductile hysteresis loops (b) Pinched hysteresis loops

FIGURE C12.1-2 Typical Hysteretic Curves

C12.1.2 Member Design, Connection Design, and Deformation Limit. Given that key elements of the seismic force-resisting system are likely to yield in response to ground motions, as discussed in Section C12.1.1, it might be expected that structural connections would be required to develop the strength of connected members. Although that is a logical procedure, it is not a general requirement. The actual requirement varies by system and generally is specified in the standards for design of the various structural materials cited by reference in Chapter 14. Good seismic design requires careful consideration of this issue.

C12.1.3 Continuous Load Path and Interconnection. In effect, Section 12.1.3 calls for the seismic design to be complete and in accordance with the principles of structural mechanics. The loads must be transferred rationally from their point of origin to the final point of resistance. This requirement should be obvious, but it often is overlooked by those inexperienced in earthquake engineering. Design consideration should be given to potentially adverse effects where there is a lack of redundancy. Given the many unknowns and uncertainties in the magnitude and characteristics of earthquake loading, in the materials and systems of construction for resisting earthquake loadings, and in the methods of analysis, good earthquake engineering practice has been to provide as much redundancy as possible in the seismic force-resisting system of buildings. Redundancy plays an important role in determining the ability of the building to resist earthquake forces. In a structural system without redundant elements, every element must remain operative to preserve the integrity of the building structure. However, in a highly redundant system, one or more redundant elements may fail and still leave a structural system that retains its integrity and can continue to resist lateral forces, albeit with diminished effectiveness.

Although a redundancy requirement is included in Section 12.3.4, overall system redundancy can be improved by making all joints of the vertical load-carrying frame moment resisting and incorporating them into the seismic force-resisting system. These multiple points of resistance can prevent a catastrophic collapse caused by distress or failure of a member or joint. (The overstrength characteristics of this type of frame are discussed in Section C12.1.1.)

The minimum connection forces are not intended to be applied simultaneously to the entire seismic force-resisting system.

C12.1.4 Connection to Supports. The requirement is similar to that given in Section 1.4 on connections to supports for general structural integrity. See Section C1.4.

C12.1.5 Foundation Design. Most foundation design criteria are still stated in terms of allowable stresses, and the forces computed in the standard are all based on the strength level of response. When developing strength-based criteria for foundations, all the factors cited in Section 12.1.5 require careful consideration. Section C12.13 provides specific guidance.

C12.1.6 Material Design and Detailing Requirements. The design limit state for resistance to an earthquake is unlike that for any other load within the scope of the standard. The earthquake limit state is based on overall system performance, not member performance, where repeated cycles of inelastic straining are accepted as an energy-dissipating mechanism. Provisions that modify customary requirements for proportioning and detailing structural members and systems are provided to produce the desired performance.

C12.2 STRUCTURAL SYSTEM SELECTION

C12.2.1 Selection and Limitations. For the purpose of seismic analysis and design requirements, seismic force-resisting systems are grouped into categories as shown in Table 12.2-1. These categories are subdivided further for various types of vertical elements used to resist seismic forces. In addition, the sections for detailing requirements are specified.

Specification of response modification coefficients, R, requires considerable judgment based on knowledge of actual earthquake performance and research studies. The coefficients and factors in Table 12.2-1 continue to be reviewed in light of recent research results. The values of R for the various systems were selected considering observed performance during past earthquakes, the toughness (ability to dissipate energy without serious degradation) of the system, and the amount of damping typically present in the system when it undergoes inelastic response. FEMA P-695 (2009b) has been developed with the purpose of establishing and documenting a methodology for quantifying seismic force-resisting system performance and response parameters for use in seismic design. Whereas R is a key parameter being addressed, related design parameters such as the overstrength factor, Ω_0, and the deflection amplification factor, C_d, also are addressed. Collectively, these terms are referred to as "seismic design coefficients (or factors)." Future systems are likely to derive their seismic design coefficients (or factors) using this methodology, and existing system coefficients (or factors) also may be reviewed in light of this new procedure.

Height limits have been specified in codes and standards for more than 50 years. The structural system limitations and limits on structural height, h_n, specified in Table 12.2-1, evolved from these initial limitations and were further modified by the collective expert judgment of the NEHRP Provisions Update Committee (PUC) and the ATC-3 project team (the forerunners of the PUC). They have continued to evolve over the past 30 years based on observations and testing, but the specific values are based on subjective judgment.

In a bearing wall system, major load-carrying columns are omitted and the walls carry a major portion of the gravity (dead and live) loads. The walls supply in-plane lateral stiffness and strength to resist wind and earthquake loads and other lateral loads. In some cases, vertical trusses are used to augment lateral stiffness. In general, lack of redundancy for support of vertical and horizontal loads causes values of R to be lower for this system compared with R values of other systems.

In a building frame system, gravity loads are carried primarily by a frame supported on columns rather than by bearing walls. Some portions of the gravity load may be carried on bearing walls, but the amount carried should represent a relatively small percentage of the floor or roof area. Lateral resistance is provided by shear walls or braced frames. Light-framed walls with shear panels are intended for use only with wood and steel building frames. Although gravity load-resisting systems are not required to provide lateral resistance, most of them do. To the extent that the gravity load-resisting system provides additional lateral resistance, it enhances the building's seismic performance capability, so long as it is capable of resisting the resulting stresses and undergoing the associated deformations.

In a moment-resisting frame system, moment-resisting connections between the columns and beams provide lateral resistance. In Table 12.2-1, such frames are classified as ordinary, intermediate, or special. In high seismic design categories, the anticipated ground motions are expected to produce large inelastic demands, so special moment frames designed and detailed for ductile response in accordance with Chapter 14 are required. In low Seismic Design Categories, the inherent overstrength in typical structural designs is such that the anticipated inelastic demands are somewhat reduced, and less ductile systems may be used safely. Because these less ductile ordinary framing systems