CHAPTER C11
SEISMIC DESIGN CRITERIA

C11.1 GENERAL

Many of the technical changes made to the seismic provisions of the 2010 edition of this standard are primarily based on Part 1 of the 2009 edition of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures*, which was prepared by the Building Seismic Safety Council (BSSC) under sponsorship of the Federal Emergency Management Agency (FEMA) as part of its contribution to the National Earthquake Hazards Reduction Program (NEHRP). The National Institute of Standards and Technology (NIST) is the lead agency for NEHRP, the federal government's long-term program to reduce the risks to life and property posed by earthquakes in the United States. Since 1985, the NEHRP Provisions have been updated every three to five years. The efforts by BSSC to produce the NEHRP provisions were preceded by work performed by the Applied Technology Council (ATC) under sponsorship of the National Bureau of Standards (NBS)—now NIST—which originated after the 1971 San Fernando Valley earthquake These early efforts demonstrated the design rules of that time for seismic resistance but had some serious shortcomings. Each subsequent major earthquake has taught new lessons. The NEHRP agencies (FEMA, NIST, NSF, USGS), ATC, BSSC, ASCE, and others have endeavored to work individually and collectively to improve each succeeding document to provide state-of-the-art earthquake engineering design and construction provisions and to ensure that the provisions have nationwide applicability.

Content of Commentary. The enhanced commentary to ASCE/SEI 7-10 is based substantially on Part 2, Commentary, of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (FEMA P-750), Building Seismic Safety Council, Federal Emergency Management Agency, 2009 edition. For additional background on the earthquake provisions contained in Chapters 11 through 23 of ASCE/SEI 7-10, the reader is referred to *Recommended Lateral Force Requirements and Commentary*, Seismology Committee, Structural Engineers Association of California, 1999.

Nature of Earthquake "Loads." Earthquakes load structures indirectly through ground motion. As the ground shakes, a structure responds. The response vibration produces structural deformations with associated strains and stresses. The computation of dynamic response to earthquake ground shaking is complex. The design forces prescribed in this standard 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 the design-level earthquake ground motion (not MCE_R).

The basic methods of analysis in the standard employ the common simplification of a response spectrum. A response spectrum for a specific earthquake ground motion provides the maximum value of response for elastic single-degree-of-freedom oscillators as a function of period without the need to reflect the total response history for every period of interest. The design response spectrum specified in Section 11.4 and used in the basic methods of analysis in Chapter 12 is a smoothed and normalized approximation for many different recorded ground motions.

The design limit state for resistance to an earthquake is unlike that for any other load within the scope of ASCE/SEI 7. The earthquake limit state is based upon system performance, not member performance, and considerable energy dissipation through repeated cycles of inelastic straining is assumed. The reason is the large demand exerted by the earthquake and the associated high cost of providing enough strength to maintain linear elastic response in ordinary buildings. This unusual limit state means that several conveniences of elastic behavior, such as the principle of superposition, are not applicable, and makes it difficult to separate design provisions for loads from those for resistance. This is the reason Chapter 14 of the standard contains so many provisions that modify customary requirements for proportioning and detailing structural members and systems. It is also the reason for the construction quality assurance requirements.

Use of Allowable Stress Design Standards. The conventional design of nearly all masonry structures and many wood and steel structures has been accomplished using allowable stress design (ASD). Although the fundamental basis for the earthquake loads in Chapters 11 through 23 is a strength limit state beyond the first yield of the structure, the provisions are written such that conventional ASD methods can be used by the design engineer. Conventional ASD methods may be used in one of two ways:

1. The earthquake load as defined in Chapters 11 through 23 may be used directly in allowable stress load combinations of Section 2.4, and the resulting stresses may be compared directly with conventional allowable stresses.
2. The earthquake load may be used in strength design load combinations and resulting stresses may be compared with amplified allowable stresses (for those materials for which the design standard gives the amplified allowable stresses, e.g., masonry).

Federal Government Construction. The Interagency Committee on Seismic Safety in Construction has prepared an order executed by the president (Executive Order 12699) that all federally owned or leased building construction, as well as federally regulated and assisted construction, should be constructed to mitigate seismic hazards and that the NEHRP provisions are deemed to be the suitable standard. It is expected

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that this standard would be deemed equivalent, but the reader should bear in mind that there are certain differences.

C11.1.1 Purpose. The purpose of Section 11.1.1 is to clarify that the detailing requirements and limitations prescribed in this section and referenced standards are still required even when the design load combinations involving the wind forces of Chapters 26 through 29 produce greater effects than the design load combinations involving the earthquake forces of Chapters 11 through 23. This is required so that the structure resists, in a ductile manner, potential seismic loads in excess of the prescribed wind loads. A proper, continuous load path is an obvious design requirement, but experience has shown that it often is overlooked and that significant damage and collapse can result. The basis for this design requirement is two-fold:

1. To ensure that the design has fully identified the seismic force-resisting system and its appropriate design level and
2. To ensure that the design basis is fully identified for the purpose of future modifications or changes in the structure.

Detailed requirements for analyzing and designing this load path are given in the appropriate design and materials chapters.

C11.1.2 Scope. Certain structures are exempt for the following reasons:

Exemption 1—Detached wood-frame dwellings not exceeding two stories above grade plane constructed in accordance with the prescriptive provisions of the International Residential Code (IRC) for light-frame wood construction, including all applicable IRC seismic provisions and limitations are deemed capable of resisting the anticipated seismic forces. Detached one- and two-story wood-frame dwellings generally have performed well even in regions of higher seismicity. Therefore, within its scope, the IRC adequately provides the level of safety required for buildings. The structures that do not meet the prescriptive limitations of the IRC are required to be designed and constructed in accordance with the International Building Code (IBC) and the ASCE/SEI-7 provisions adopted therein.

Exemption 2—Agricultural storage structures generally are exempt from most code requirements because such structures are intended only for incidental human occupancy and represent an exceptionally low risk to human life.

Exemption 3—Bridges, transmission towers, nuclear reactors, and other structures with special configurations and uses are not covered. The regulations for buildings and building-like structures presented in this document do not adequately address the design and performance of such special structures.

ASCE/SEI 7-10 is not retroactive and usually applies to existing structures only when there is an addition, change of use, or alteration. Minimum acceptable seismic resistance of existing buildings is a policy issue normally set by the authority having jurisdiction. Appendix 11B of the standard contains rules of application for basic conditions. ASCE/SEI 31, *Seismic Evaluation of Existing Buildings* and ASCE/SEI 41, *Seismic Rehabilitation of Existing Buildings*, provide technical guidance but do not contain policy recommendations. A chapter in the *International Building Code* (IBC) applies to alteration, repair, addition, and change of occupancy of existing buildings, and the International Code Council maintains the International Existing Building Code (IEBC) and associated commentary.

C11.1.3 Applicability. Industrial buildings may be classified as nonbuilding structures in certain situations for the purposes of determining seismic design coefficients and factors, system limitations, height limits, and associated detailing requirements. Many industrial building structures have geometries and/or framing systems that are different from the broader class of occupied structures addressed by Chapter 12, and the limited nature of the occupancy associated with these buildings reduces the hazard associated with their performance in earthquakes. Therefore, when the occupancy is limited primarily to maintenance and monitoring operations, these structures may be designed in accordance with the provisions of Section 15.5 for nonbuilding structures similar to buildings. Examples of such structures include, but are not limited to, boiler buildings, aircraft hangars, steel mills, aluminum smelting facilities, and other automated manufacturing facilities, whereby the occupancy restrictions for such facilities should be uniquely reviewed in each case. These structures may be clad or open structures.

C11.1.4 Alternate Materials and Methods of Construction. It is not possible for a design standard to provide criteria for the use of all possible materials and their combinations and methods of construction, either existing or anticipated. This section serves to emphasize that the evaluation and approval of alternate materials and methods require a recognized and accepted approval system. The requirements for materials and methods of construction contained within the document represent the judgment of the best use of the materials and methods based on well established expertise and historical seismic performance. It is important that any replacement or substitute be evaluated with an understanding of all the ramifications of performance, strength, and durability implied by the standard.

Until needed standards and agencies are created, authorities that have jurisdiction need to operate on the basis of the best evidence available to substantiate any application for alternates. If accepted standards are lacking, applications for alternative materials or methods should be supported by test data obtained from tests data requirements in Section 1.3.1.2. The tests should simulate expected load and deformation conditions to which the system, component, or assembly may be subjected during the service life of the structure. These conditions, when applicable, should include several cycles of full reversals of loads and deformations in the inelastic range.

C11.2 DEFINITIONS

ATTACHMENTS, COMPONENTS, AND SUPPORTS: The distinction between attachments, components, and supports is necessary to the understanding of the requirements for nonstructural components and nonbuilding structures. Common cases associated with nonstructural elements are illustrated in Fig. C11-1. The definitions of components, supports, and attachments are generally applicable to components with a defined envelope in the as-manufactured condition and for which additional supports and attachments are required to provide support in the as-built condition. This distinction may not always be clear, particularly when the component is equipped with prefabricated supports; therefore, judgment must be used in the assignment of forces to specific elements in accordance with the provisions of Chapter 13.

BASE: The following factors affect the location of the seismic base:

- location of the grade relative to floor levels,
- soil conditions adjacent to the building,
- openings in the basement walls,
- location and stiffness of vertical elements of the seismic force-resisting system,
- location and extent of seismic separations,
- depth of basement,

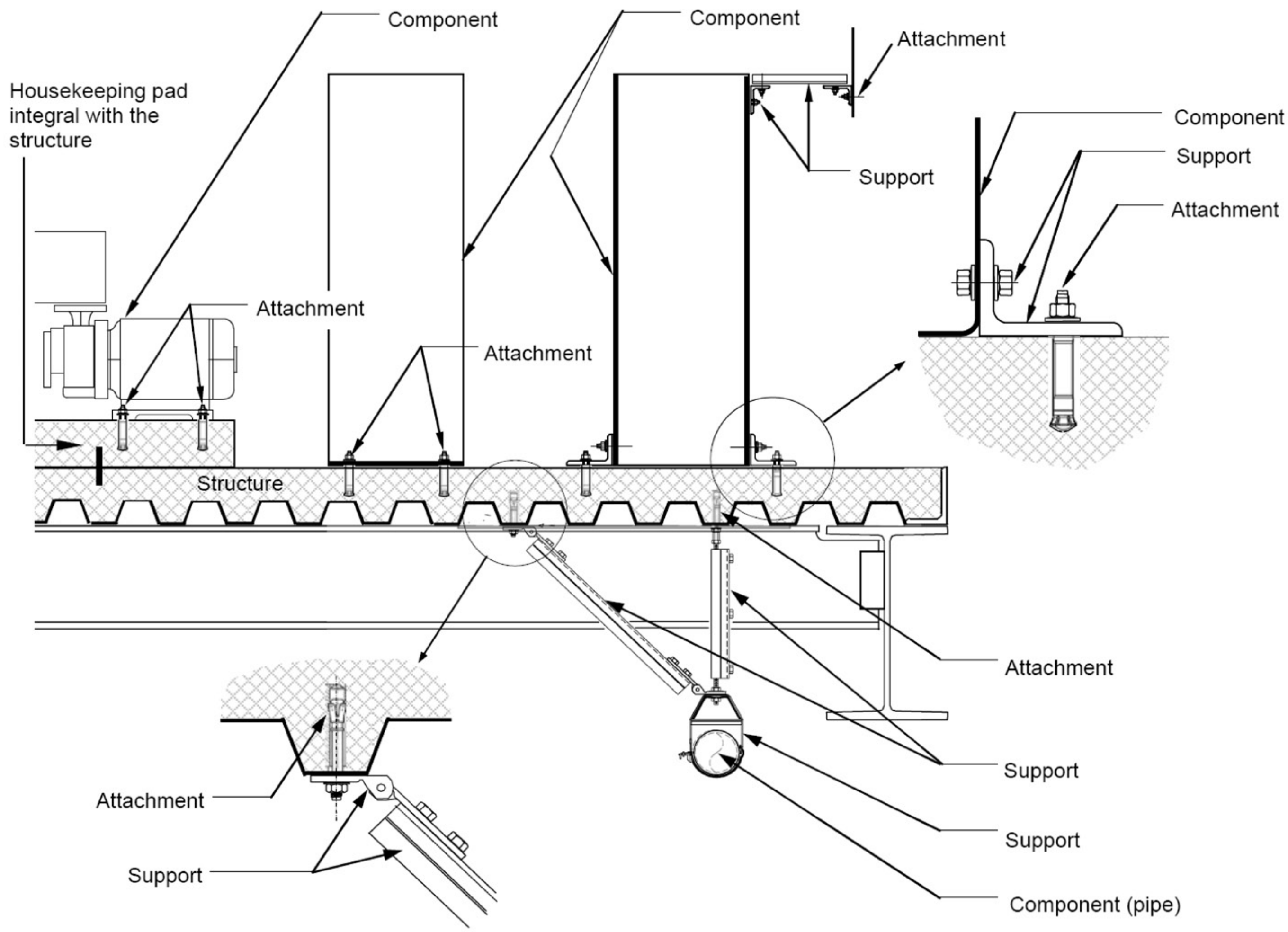

FIGURE C11-1 Examples of Components, Supports, and Attachments

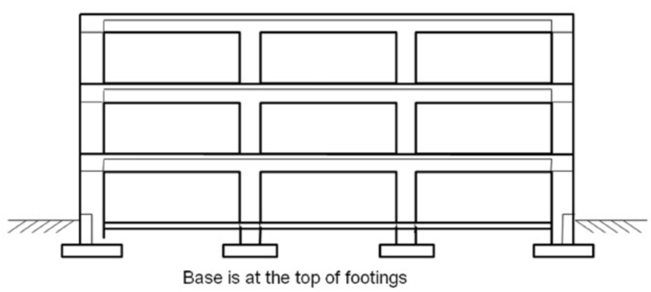

FIGURE C11-2 Base for a Level Site

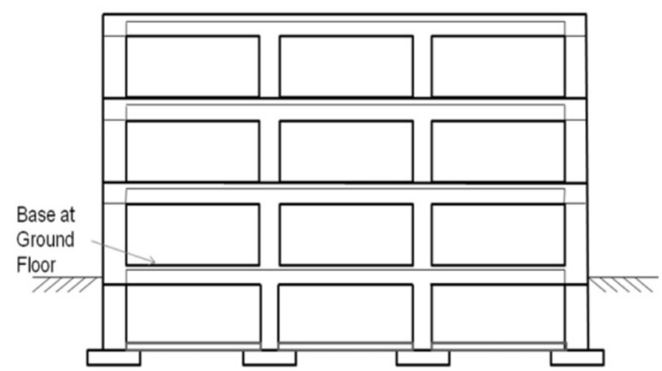

FIGURE C11-3 Base at Ground Floor Level

- manner in which basement walls are supported,
- proximity to adjacent buildings, and
- slope of grade.

For typical buildings on level sites with competent soils, the base is generally close to the grade plane. For a building without a basement, the base is generally established near the ground-level slab elevation as shown in Fig. C11-2. Where the vertical elements of the seismic force-resisting system are supported on interior footings or pile caps, the base is the top of these elements. Where the vertical elements of the seismic force-resisting system are supported on top of perimeter foundation walls, the base is typically established at the top of the foundation walls. Often vertical elements are supported at various elevations on the top of footings, pile caps, and perimeter foundation walls. Where this occurs, the base is generally established as the lowest elevation of the tops of elements supporting the vertical elements of the seismic force-resisting system.

For a building with a basement located on a level site, it is often appropriate to locate the base at the floor closest to grade, as shown in Fig. C11-3. If the base is to be established at the level located closest to grade, the soil profile over the depth of the basement should not be liquefiable in the MCE_G ground

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motion. The soil profile over the depth of the basement also should not include quick and highly sensitive clays or weakly cemented soils prone to collapse in the MCE_G ground motion. Where liquefiable soils or soils susceptible to failure or collapse in an MCE_G ground motion are located within the depth of the basement, the base may need to be located below these soils rather than close to grade. Stiff soils are required over the depth of the basement because seismic forces will be transmitted to and from the building at this level and over the height of the basement walls. The engineer of record is responsible for establishing whether the soils are stiff enough to transmit seismic forces near grade. For tall or heavy buildings or where soft soils are present within the depth of the basement, the soils may compress laterally too much during an earthquake to transmit seismic forces near grade. For these cases, the base should be located at a level below grade.

In some cases, the base may be at a floor level above grade. For the base to be located at a floor level above grade, stiff foundation walls on all sides of the building should extend to the underside of the elevated level considered the base. Locating the base above grade is based on the principles for the two-stage equivalent lateral force procedure for a flexible upper portion of a building with one-tenth the stiffness of the lower portion of the building as permitted in Section 12.2.3.2. For a floor level above grade to be considered the base, it generally should not be above grade more than one-half the height of the basement story, as shown in Fig. C11-4. Figure C11-4 illustrates the concept of the base level located at the top of a floor level above grade, which also includes light-frame floor systems that rest on top of stiff basement walls or stiff crawl space stem walls of concrete or masonry construction.

A condition where the basement walls that extend above grade on a level site may not provide adequate stiffness is where the basement walls have many openings for items such as light wells, areaways, windows, and doors, as shown in Fig. C11-5.

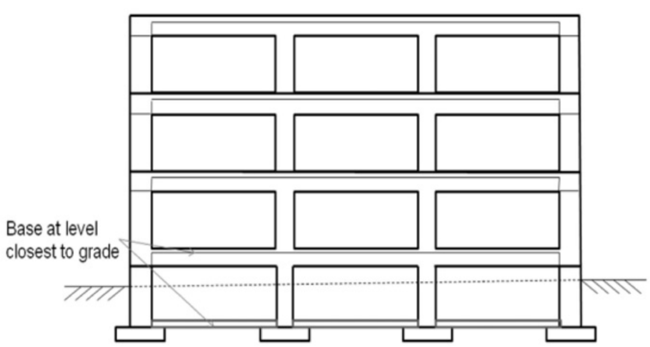

FIGURE C11-4 Base at Level Closest to Grade Elevation

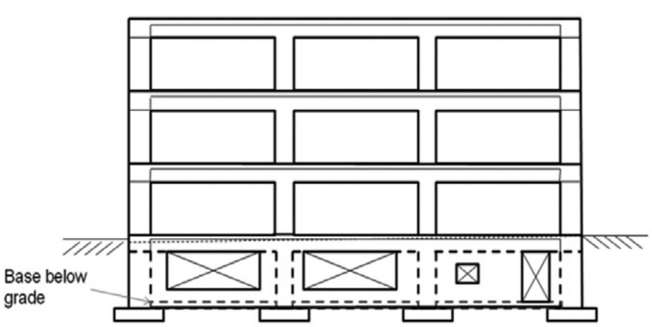

FIGURE C11-5 Base Below Substantial Openings in Basement Wall

Where the basement wall stiffness is inadequate, the base should be taken as the level close to but below grade. If all of the vertical elements of the seismic force-resisting system are located on top of basement walls and there are many openings in the basement walls, it may be appropriate to establish the base at the bottom of the openings. Another condition where the basement walls may not be stiff enough is where the vertical elements of the seismic force-resisting system are long concrete shear walls extending over the full height and length of the building, as shown in Fig. C11-6. For this case, the appropriate location for the base is the foundation level of the basement walls.

Where the base is established below grade, the weight of the portion of the story above the base that is partially above and below grade must be included as part of the effective seismic weight. If the equivalent lateral force procedure is used, this can result in disproportionately high forces in upper levels due to a large mass at this lowest level above the base. The magnitude of these forces can often be mitigated by using the two-stage equivalent lateral force procedure where it is allowed, or by using dynamic analysis to determine force distribution over the height of the building. If dynamic analysis is used, it may be necessary to include multiple modes to capture the required mass participation, unless soil springs are incorporated into the model. Incorporation of soil springs into the model generally reduces seismic forces in the upper levels. With one or more stiff stories below more flexible stories, the dynamic behavior of the structure may result in the portion of the base shear from the first mode being less than the portion of base shear from higher modes.

Other conditions may also necessitate establishing the base below grade for a building with a basement that is located on a level site. Such conditions include those where seismic separations extend through all floors, including those located close to and below grade; those where the floor diaphragms close to and below grade are not tied to the foundation wall; those where the floor diaphragms, including the diaphragm for the floor close to grade, are flexible; and those where other buildings are located nearby.

For a building with seismic separations extending through the height of the building including levels close to and below grade, the separate structures will not be supported by the soil against a basement wall on all sides in all directions. If there is only one joint through the building, assigning the base to the level close to grade may still be appropriate if the soils over the depth of the basement walls are stiff and the diaphragm is rigid. Stiff soils are required so that the seismic forces can be transferred between the soils and basement walls in both bearing and side friction. If the soils are not stiff, adequate side friction may not develop for movement in the direction perpendicular to the joint.

For large footprint buildings, seismic separation joints may extend through the building in two directions and there may be

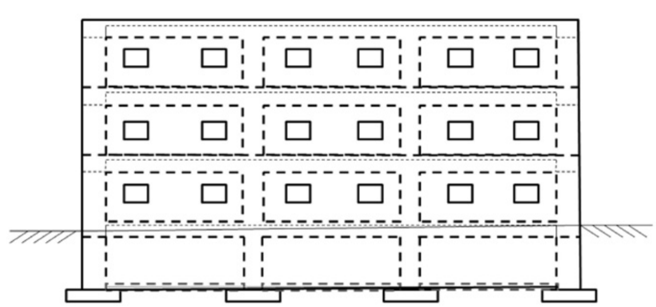

FIGURE C11-6 Base at Foundation Level Where There Are Full-Length Exterior Shear Walls

multiple parallel joints in a given direction. For individual structures within these buildings, substantial differences in the location of the center of rigidity for the levels below grade relative to levels above grade can lead to torsional response. For such buildings, the base should usually be at the foundation elements below the basement or the highest basement slab level where the separations are no longer provided.

Where floor levels are not tied to foundation walls, the base may need to be located well below grade at the foundation level. An example is a building with tie-back walls and post-tensioned floor slabs. For such a structure, the slabs may not be tied to the wall to allow relative movement between them. In other cases a soft joint may be provided. If shear forces cannot be transferred between the wall and a ground level or basement floor, the location of the base will depend on whether forces can be transferred through bearings between the floor diaphragm and basement wall and between the basement wall and the surrounding soils. Floor diaphragms bearing against the basement walls must resist the compressive stress from earthquake forces without buckling. If a seismic or expansion joint is provided in one of these buildings, the base will almost certainly need to be located at the foundation level or a level below grade where the joint no longer exists.

If the diaphragm at grade is flexible and does not have substantial compressive strength, the base of the building may need to be located below grade. This condition is more common with existing buildings. Newer buildings with flexible diaphragms should be designed for compression to avoid the damage that will otherwise occur.

Proximity to other structures can also affect where the base should be located. If other buildings with basements are located adjacent to one or more sides of a building, it may be appropriate to locate the base at the bottom of the basement. The closer the adjacent building is to the building, the more likely it is that the base should be below grade.

For sites with sloping grade, many of the same considerations for a level site are applicable. For example, on steeply sloped sites the earth may be retained by a tie-back wall so that the building does not have to resist the lateral soil pressures. For such a case, the building will be independent of the wall, so the base should be located at a level close to the elevation of grade on the side of the building where it is lowest, as shown in Fig. C11-7. Where the building's vertical elements of the seismic force-resisting system also resist lateral soil pressures, as shown in Fig. C11-8, the base should also be located at a level close to the elevation of grade on the side of the building where grade is low. For these buildings, the seismic force-resisting system below highest grade is often much stiffer than the system used above it, as shown in Fig. C11-9, and the seismic weights for levels close to and below highest grade are greater than for levels above highest grade. Use of a two-stage equivalent lateral force procedure can be useful for these buildings.

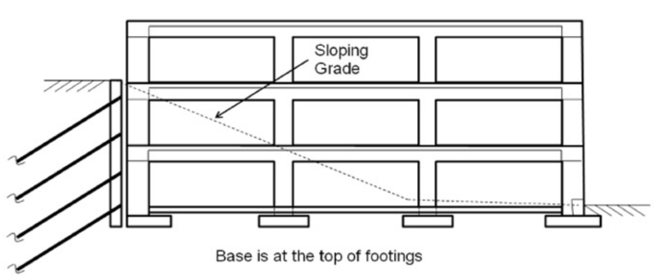

FIGURE C11-7 Building with Tie-Back or Cantilevered Retaining Wall that Is Separate from the Building

Where the site is moderately sloped such that it does not vary in height by more than a story, stiff walls often extend to the underside of the level close to the elevation of high grade, and the seismic force-resisting system above grade is much more flexible above grade than it is below grade. If the stiff walls extend to the underside of the level close to high grade on all sides of the building, locating the base at the level closest to high grade may be appropriate. If the stiff lower walls do not extend to the underside of the level located closest to high grade on all sides of the building, the base should be assigned to the level closest to low grade. If there is doubt as to where to locate the base, it should conservatively be taken at the lower elevation.

STORY ABOVE GRADE PLANE. Figure C11-10 illustrates this definition.

C11.4 SEISMIC GROUND MOTION VALUES

The seismic ground motion values of Section 11.4 are defined by 0.2 s and 1 s spectral response accelerations (5% of critical damping) for the mapped values of risk-targeted maximum considered earthquake (MCE_R) ground motions provided in Chapter 22. The U.S. Geological Survey (USGS) prepared the mapped values of MCE_R ground motions of Chapter 22 in accordance with (1) the site-specific ground motion procedures of Section 21.2, (2) updates of the United States National Seismic Hazard Maps (Petersen et. al. 2008), and (3) results of related research (Huang et al. 2008). The mapped values of MCE_R ground motion parameters S_s and S_1 are derived as the lesser of the probabilistic MCE_R spectral response acceleration (Section 21.2.1) and the deterministic MCE_R spectral response acceleration (Section 21.2.2) at any location. The deterministic MCE_R spectral response acceleration has a lower limit as shown in Fig. 21.2-1. The USGS calculated the probabilistic (MCE_R) spectral response acceleration using the iterative procedure (Method 2) of Section 21.2.1.2

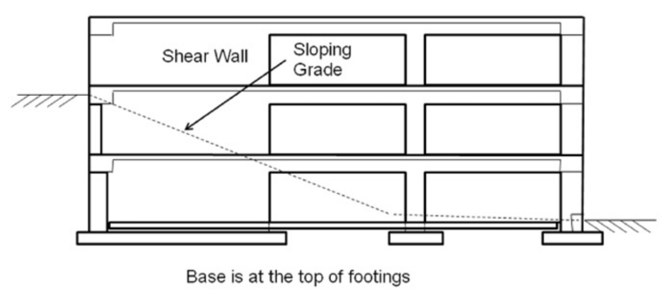

FIGURE C11-8 Building with Vertical Elements of the Seismic Force-Resisting System Supporting Lateral Earth Pressures

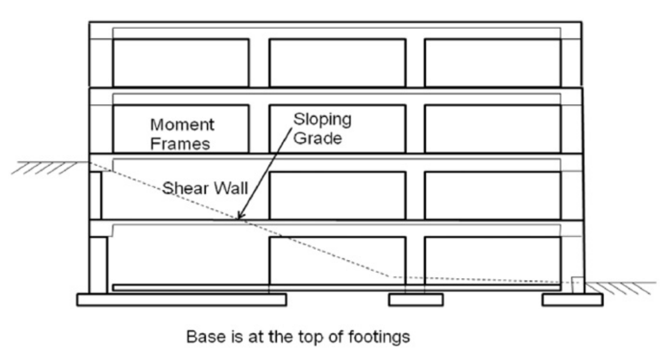

FIGURE C11-9 Building with Vertical Elements of the Seismic Force-Resisting System Supporting Lateral Earth Pressures

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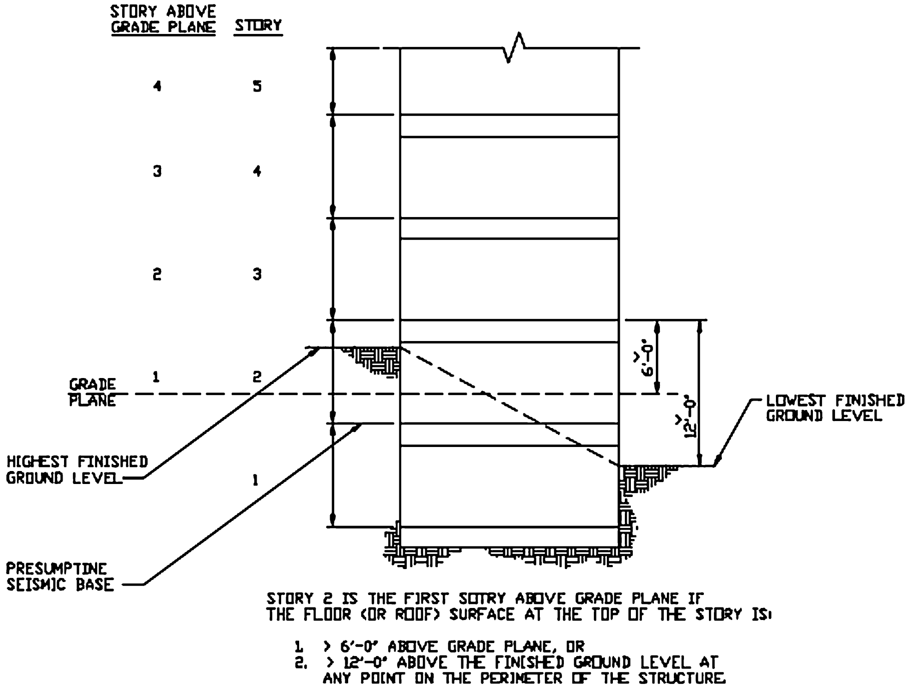

FIGURE C11-10 Illustration of Definition of Story above Grade Plane

(Luco et al. 2007). Mapped values of MCE_R ground motions are governed by probabilistic MCE_R response spectral acceleration, except at high hazard sites located relatively close to an active fault.

The basis for the mapped values of the MCE_R ground motions in ASCE/SEI 7-10 is significantly different from that of the mapped values of MCE ground motions in previous editions of ASCE/SEI 7. These differences include use of (1) probabilistic ground motions that are based on uniform collapse risk, rather than uniform hazard; (2) deterministic ground motions that are based on the 84th percentile (approximately 1.8 times median), rather than 1.5 times median response spectral acceleration for sites near active faults; and (3) ground motion intensity that is based on maximum, rather than the average (geometrical mean), response spectra acceleration in the horizontal plane. Except for determining the MCE_G peak ground acceleration (PGA) values in Chapters 11 and 21, the mapped values are given as MCE_R spectral response accelerations.

The approach adopted in Section 11.4 is intended to provide for a more uniform collapse risk for structures designed using the MCE_R ground motions. The MCE_R ground motions are expected to result in structures with a 1% probability of collapse in 50 years, based on the probabilistic seismic hazard at each site and a probabilistic estimate of the margin against collapse inherent in structures designed to the seismic provisions in the standard (collapse fragility). In previous editions of ASCE/SEI 7, the lower bound margin was judged, based on experience, to correspond to a factor of about 1.5 in ground motions. In ASCE/SEI 7-10 the uncertainty in this margin is accounted for with the collapse fragility defined in Section 21.2.1.2. Nevertheless, the design earthquake ground motion is based on 1/1.5 (or 2/3) of MCE_R ground motion for consistency with previous editions of the standard. This factor has been taken into account in developing the MCE_R ground motions.

Probabilistic (MCE_R) ground motions are based on the assumption that buildings designed in accordance with ASCE/SEI 7-10 have a collapse probability of not more than 10%, on average, if MCE_R ground motions occur at the building site. The conditional probability of 10% is an idealized collapse safety goal of FEMA P-695 (FEMA 2009). The FEMA P-695 study investigated the collapse probability of a limited number of different types of seismic force-resisting systems and found that systems designed in accordance with ASCE/SEI 7-10 generally conform to the 10% collapse safety goal. While stronger shaking could occur, it was judged economically impractical to design for such ground motions and that MCE_R ground motions based on a 1% probability of collapse in 50 years provides an acceptable level of seismic safety.

In regions of high seismicity, such as in many areas of California, the seismic hazard is typically controlled by large-magnitude events occurring on a limited number of well defined fault systems. Probabilistic ground motions calculated for a 1% probability of collapse in 50 years can be significantly larger than deterministic ground motions based on the characteristic magnitudes of earthquakes on these known active faults. Probabilistic ground motions tend to be greater when major active faults produce characteristic earthquakes every few hundred years, rather than at longer return periods. For these regions, it is considered more appropriate to determine MCE_R ground motions directly by deterministic methods based on a conservative estimate of the ground shaking associated with characteristic earthquakes of well defined fault systems. To provide an appropriate level of conservatism in the design process, 84th percentile ground motions are used to define deterministic (MCE_R) ground motion, which is estimated as median ground motions for characteristic events multiplied by 1.8 (Huang et al. 2008).

The smaller deterministic (MCE_R) ground motions result in a probability of collapse in 50 years greater than the targeted 1% of probabilistic (MCE_R) ground motions, but the assumption of not more than a 10% probability of collapse if the MCE_R ground motion occurs at the building site still applies.

It was judged economically impractical to design buildings in high seismic regions located near active faults for more than 84th percentile ground motions of characteristic earthquakes, and that a 10% probability of collapse if these ground motions occur at the building site provides an acceptable level of safety.

C11.4.1 Mapped Acceleration Parameters. In the general procedure, seismic design values are computed from mapped values of the spectral response acceleration at short periods, S_s, and at 1 s, S_1, for Class B sites. These S_s and S_1 values may be obtained directly from Figs. 22-1 through 22-6 (in Chapter 22). However, these maps themselves do not permit precise determination of S_s and S_1 values, especially in high seismic regions, so the mapped values may be obtained directly from the USGS website: http://earthquake.usgs.gov/designmaps, or through the ASCE Structural Engineering Institute website: http://content.seinstitute.org.

S_s is the mapped value of the 5% damped MCE_R spectral response acceleration for short-period structures founded on Site Class B (firm rock) sites. The short-period acceleration has been determined at a period of 0.2 s because it was concluded that 0.2 s was reasonably representative of the shortest effective period of buildings and structures that are designed using the standard, considering the effects of soil compliance, foundation rocking, and other factors typically neglected in structural analysis.

Similarly, S_1 is the mapped value of the 5% damped MCE_R spectral response acceleration at a period of 1 s on a Site Class B. The spectral response acceleration at periods other than 1 s typically can be derived from the acceleration at 1 s. Consequently, for MCE_R ground shaking on Site Class B sites, these two response acceleration parameters, S_s and S_1, are sufficient when adjusted for site effects to define an entire response spectrum for the period range of importance for most buildings and structures using the generic shape of the MCE_R response spectrum shown in Fig. 21.2-1.

C11.4.3 Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters. Using the general procedure to obtain acceleration response parameters that are appropriate for sites with a classification other than Site Class B, the S_s and S_1 values must be modified as indicated in Section 11.4.3. This modification is performed using two coefficients, F_a and F_v, that respectively scale the S_s and S_1 values determined for Site Class B to values appropriate for other site classes. The MCE_R spectral response accelerations adjusted for site class are designated S_{MS} and S_{M1}, respectively, for short-period and 1-s-period response.

The site coefficients, F_a and F_v, presented respectively in Tables 11.4-1 and 11.4-2 for the various site classes, are based on the results of empirical analyses of strong-motion data and analytical studies of site response.

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 greater amplifications than stiffer soils with higher shear-wave 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 shorter periods.

An extensive discussion of the development of the F_a and F_v 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, earthquake events 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), Harmsen (1997), Dobry et al. (1999), Silva et al. (2000), Joyner and Boore (2000), Field (2000), Steidl (2000), Rodriguez-Marek et al. (2001), Borcherdt (2002), and Stewart et al. (2003). Although the results of these studies vary, the site amplification factors are generally consistent with those in Tables 11.4-1 and 11.4-2.

C11.4.4 Design Spectral Acceleration Parameters. As described in Section C11.4, structural design in ASCE/SEI 7-10 is performed for earthquake demands that are 2/3 of the MCE_R response spectra. As set forth in Section 11.4.4, two additional parameters, S_{DS} and S_{D1}, are used to define the acceleration response spectrum for this design level event. These parameters are 2/3 of the respective S_{MS} and S_{M1} values and define a design response spectrum for sites of any characteristics and for natural periods of vibration less than the transition period, T_L. Values of S_{MS}, S_{M1}, S_{DS}, and S_{D1} can also be obtained from the USGS website cited previously.

C11.4.5 Design Response Spectrum. The design response spectrum (Fig. 11.4-1) consists of several segments. The constant-acceleration segment covers the period band from T_0 to T_s; response accelerations in this band are constant and equal to S_{DS}. The constant-velocity segment covers the period band from T_s to T_L, and the response accelerations in this band are proportional to $1/T$ with the response acceleration at 1-sec period equal to S_{D1}. The long-period portion of the design response spectrum is defined on the basis of the parameter, T_L, the period that marks the transition from the constant-velocity segment to the constant-displacement segment of the design response spectrum. Response accelerations in the constant-displacement segment, where $T \geq T_L$, are proportional to $1/T^2$. Values of T_L are provided on maps in Figs. 22-12 through 22-16.

The T_L maps were prepared following a two-step procedure. First, a correlation between earthquake magnitude and T_L was established. Then, the modal magnitude from deaggregation of the ground-motion seismic hazard at a 2-s period (a 1-s period for Hawaii) was mapped. Details of the procedure and the rationale for it are found in Crouse et al. (2006).

C11.4.7 Site-Specific Ground Motion Procedures. Site-specific ground motions are permitted for design of any structure and required for design of certain structures and certain site soil conditions. The objective of a site-specific ground-motion analysis is to determine ground motions for local seismic and site conditions with higher confidence than is possible using the general procedure of Section 11.4.

As noted earlier, the site-specific procedures of Chapter 21 are the same as those used by the USGS to develop the mapped values of MCE_R ground motions shown in Figs. 22-1 through 22-6 of Chapter 22. Unless significant differences in local seismic and site conditions are determined by a site-specific analysis of earthquake hazard, site-specific ground motions would not be expected to differ significantly from those of the mapped values of MCE_R ground motions prepared by the USGS.

C11.5 IMPORTANCE FACTOR AND RISK CATEGORY

Large earthquakes are rare events that include severe ground motions. Such events are expected to result in damage to structures even if they were designed and built in accordance with the minimum requirements of the standard. The consequence of structural damage or failure is not the same for the various types of structures located within a given community. Serious damage to certain classes of structures, such as critical facilities (e.g., hospitals), will disproportionally affect a community. The fundamental purpose of this section and of subsequent requirements that depend on this section is to improve the ability of a community to recover from a damaging earthquake by tailoring the seismic protection requirements to the relative importance of a structure. That purpose is achieved by requiring improved performance for structures that

1. Are necessary to response and recovery efforts immediately following an earthquake,
2. Present the potential for catastrophic loss in the event of an earthquake, or
3. House a very large number of occupants or occupants less able to care for themselves than the average.

The first basis for seismic design in the standard is that structures will have a suitably low likelihood of collapse in rare events defined as the maximum considered earthquake (MCE) ground motion. A second basis is that life-threatening damage, primarily from failure of nonstructural components in and on structures, will be unlikely in a design earthquake ground motion (defined as two-thirds of the MCE). Given the occurrence of ground motion equivalent to the MCE, a population of structures built to meet these design objectives will probably still experience substantial damage in many structures, rendering these structures unfit for occupancy or use. Experience in past earthquakes around the world has demonstrated that there will be an immediate need to treat injured people, to extinguish fires and prevent conflagration, to rescue people from severely damaged or collapsed structures, and to provide shelter and sustenance to a population deprived of its normal means. These needs are best met when structures essential to response and recovery activities remain functional.

The standard addresses these objectives by requiring that each structure be assigned to one of the four risk categories presented in Chapter 1 and by assigning an importance factor, I_e, to the structure based on that risk category. (The two lowest categories, I and II are combined for all purposes within the seismic provisions.) The risk category is then used as one of two components in determining the seismic design category (see Section C11.6) and is a primary factor in setting drift limits for building structures under the design earthquake ground motion (see Section C12.12).

Figure C11-11 shows the combined intent of these requirements for design. The vertical scale is the likelihood of the ground motion with the MCE being the rarest considered. The horizontal scale is the level of performance intended for the structure and attached nonstructural components, which range from collapse to operational. The basic objective of collapse prevention at the MCE for ordinary structures (Risk Category II) is shown at the lower right by the solid triangle; protection from life-threatening damage at the design earthquake ground motion (defined by the standard as two-thirds of the MCE) is shown by the open triangle. The performance implied for higher risk categories III and IV is shown by squares and circles, respectively. The performance anticipated for less severe ground motion is shown by dotted symbols.

C11.5.1 Importance Factor I_e. The importance factor, I_e, is used throughout the standard in quantitative criteria for strength. In most of those quantitative criteria, the importance factor is shown as a divisor on the factor R or R_p to reduce damage for important structures in addition to preventing collapse in larger ground motions. The R and R_p factors adjust the computed linear elastic response to a value appropriate for design; in many structures, the largest component of that adjustment is ductility (the ability of the structure to undergo repeated cycles of inelastic strain in opposing directions). For a given strength demand, reducing the effective R factor (by means of the importance factor) increases the required yield strength, thus reducing ductility demand and related damage.

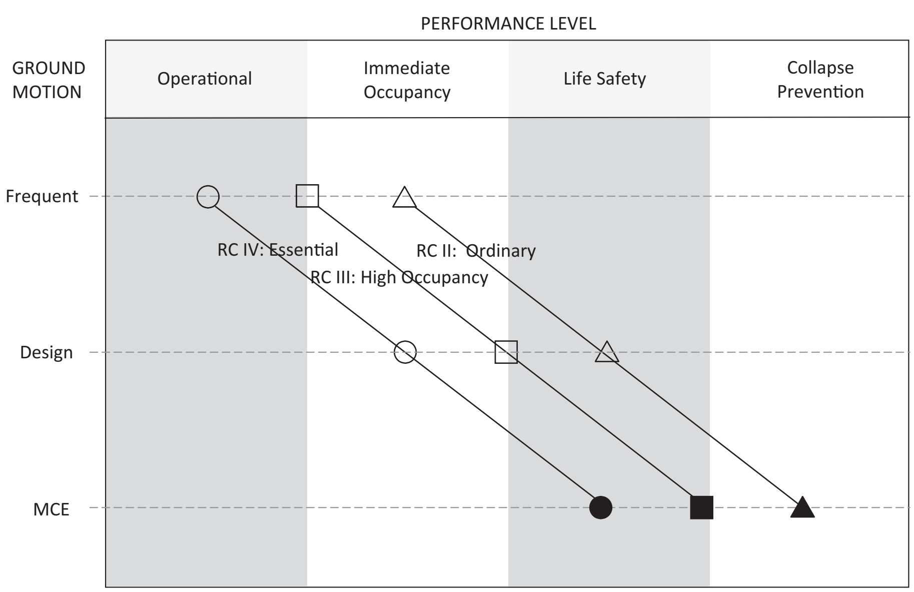

FIGURE C11-11 Expected Performance as Related to Risk Category (RC) and Level of Ground Motion

C11.5.2 Protected Access for Risk Category IV. Those structures considered essential facilities for response and recovery efforts must be accessible to carry out their purpose. For example, if the collapse of a simple canopy at a hospital could block ambulances from the emergency room admittance area, then the canopy must meet the same structural standard as the hospital. The protected access requirement must be considered in the siting of essential facilities in densely built urban areas.

C11.6 SEISMIC DESIGN CATEGORIES

Seismic design categories (SDCs) provide a means to step progressively from simple, easily performed design and construction procedures and minimums to more sophisticated, detailed, and costly requirements as both the level of seismic hazard and the consequence of failure escalate. The SDCs are used to trigger requirements that are not scalable; such requirements are either on or off. For example, the basic amplitude of ground motion for design is scalable—the quantity simply increases in a continuous fashion as one moves from a low hazard area to a high hazard area. However, a requirement to avoid weak stories is not particularly scalable. Requirements such as this create step functions. There are many such requirements in the standard, and the SDCs are used systematically to group these step functions. (Further examples include whether seismic anchorage of nonstructural components is required or not, whether particular inspections will be required or not, and structural height limits applied to various seismic force-resisting systems.)

In this regard, SDCs perform one of the functions of the seismic zones used in earlier U.S. building. However, SDCs also depend on a building's occupancy and, therefore, its desired performance. Furthermore, unlike the traditional implementation of seismic zones, the ground motions used to define the SDCs include the effects of individual site conditions on probable ground-shaking intensity.

In developing the ground motion limits and design requirements for the various seismic design categories, the equivalent modified Mercalli intensity (MMI) scale was considered. There are now correlations of the qualitative MMI scale with quantitative characterizations of ground motions. The reader is encouraged to consult any of a great many sources that describe the MMIs. The following list is a very coarse generalization:

MMI V	No real damage
MMI VI	Light nonstructural damage
MMI VII	Hazardous nonstructural damage
MMI VIII	Hazardous damage to susceptible structures
MMI IX	Hazardous damage to robust structures

When the current design philosophy was adopted (the 1997 edition of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, FEMA 302, and *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2, Commentary*, FEMA 303), the upper limit for SDC A was set at roughly one-half of the lower threshold for MMI VII and the lower limit for SDC D was set at roughly the lower threshold for MMI VIII. However, the lower limit for SDC D was more consciously established by equating that design value (two-thirds of the MCE) to one-half of what had been the maximum design value in building codes over the period of 1975 to 1995. As more correlations between MMI and numerical representations of ground motion have been created, it is reasonable to make the following correlation between the MMI at MCE ground motion and the seismic design category (all this discussion is for ordinary occupancies):

MMI V	SDC A
MMI VI	SDC B
MMI VII	SDC C
MMI VIII	SDC D
MMI IX	SDC E

An important change was made to the determination of SDC when the current design philosophy was adopted. Earlier editions of the *NEHRP Provisions* used 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 NEHRP provisions adopted the use of response spectral acceleration parameters S_{DS} and S_{D1}, which include site soil effects for this purpose.

Except for the lowest level of hazard (SDC A), the SDC also depends on the risk categories. For a given level of ground motion, the SDC is one category higher for Risk Category IV structures than for lower-risk structures. This has the effect of increasing the confidence that the design and construction requirements will deliver the intended performance in the extreme event.

Note that the tables in the standard are at the design level, defined as two-thirds of the MCE level. Also recall that the MMIs are qualitative by their nature, and that the above correlation will be more or less valid, depending on which numerical correlation for MMI is used. The numerical correlations for MMI roughly double with each step so correlation between design earthquake ground motion and MMI is not as simple or convenient.

In sum, at the MCE level, SDC A structures should not see motions that are normally destructive to structural systems, whereas the MCE level motions for SDC D structures can destroy vulnerable structures. The grouping of step function requirements by SDC is such that there are a few basic structural integrity requirements imposed at SDC A graduating to a suite of requirements at SDC D based on observed performance in past earthquakes, analysis, and laboratory research.

The nature of ground motions within a few kilometers of a fault can be very different from more distant motions. For example, some near-fault motions will have strong velocity pulses, associated with forward rupture directivity, that tend to be highly destructive to irregular structures, even if they are well detailed. For ordinary occupancies, the boundary between SDCs D and E is set to define sites likely to be close enough to a fault that these unusual ground motions may be present. Note that this boundary is defined in terms of mapped bedrock outcrop motions affecting response at 1 s, not site-adjusted values, to better discriminate between sites near and far from faults. Short-period response is not normally as affected as the longer period response. The additional design criteria imposed on structures in SDCs E and F specifically are intended to provide acceptable performance under these very intense near-fault ground motions.

For most buildings, the SDC is determined without consideration of the building's period. Structures are assigned to an SDC based on the more severe condition determined from 1-second acceleration and short-period acceleration. This is done for several reasons. Perhaps the most important of these is that it is often difficult to estimate precisely the period of a structure using default procedures contained in the standard. Consider, for example, the case of rigid wall/flexible diaphragm buildings,

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including low-rise reinforced masonry and concrete tilt-up buildings with either untopped metal deck or wood diaphragms. The formula in the standard for determining the period of vibration of such buildings is based solely on the structural height, h_n, and the length of wall present. These formulas typically indicate very short periods for such structures, often on the order of 0.2 s or less. However, the actual dynamic behavior of these buildings often is dominated by the flexibility of the diaphragm—a factor neglected by the formula for approximate fundamental period. Large buildings of this type can have actual periods on the order of 1 s or more. To avoid misclassifying a building's SDC by inaccurately estimating the fundamental period, the standard generally requires that the more severe SDC determined on the basis of short- and long-period shaking be used.

Another reason for this requirement is a desire to simplify building regulation by requiring all buildings on a given soil profile in a particular region to be assigned to the same SDC, regardless of the structural type. This has the advantage of permitting uniform regulation in the selection of seismic force-resisting systems, inspection and testing requirements, seismic design requirements for nonstructural components, and similar aspects of the design process regulated on the basis of SDC, within a community.

Notwithstanding the above, it is recognized that classification of a building as SDC C instead of B or D can have a significant impact on the cost of construction. Therefore, the standard includes an exception permitting the classification of buildings that can reliably be classified as having short structural periods on the basis of short-period shaking alone.

Local or regional jurisdictions enforcing building regulations may desire 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 a historical practice of high seismic zone detailing might mandate a minimum SDC 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 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.

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 1.4.3 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 1.4.3) and a minimum strength for connections of structural members (Section 1.4.4).

For many buildings, the wind force will control 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 1.4.3 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 (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 1.4.4 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 1.4.5). 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 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 for Site Class B bedrock (PGA), plus a site-coefficient (F_{PGA}) table to convert the PGA value to one adjusted for site class effects (PGA_M).

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 peak ground acceleration for bedrock Site Class B (PGA) 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 improves 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 (CEUS), 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.

Maps of MCE_G PGA for Site Class B bedrock, similar to maps of S_s and S_1, are shown in Figs. 22-7 to 22-11 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 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 values of F_{PGA} in Table 11.8-1 are identical to the F_a values in Table 11.4-1. These F_a values are the same as those in ASCE 7-05 and ASCE 7-02. The method of determining site class, used in the determination of F_a, 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 have been derived for the "maximum direction shaking" and are now 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 assure 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 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 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 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, now require 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 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 (see Fig. C11-11). 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 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

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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 now 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. 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, etc.) 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 build up 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.

REFERENCES

Borcherdt, R. D. (2002). "Empirical evidence for site coefficients in building-code provisions." *Earthquake Spectra*, 18(2), 189–217.

Crouse, C. B., and McGuire, J. W. (1996). "Site response studies for purposes of revising NEHRP seismic provisions." *Earthquake Spectra*, 12(3).

Crouse, C. B., Leyendecker, E. V., Somerville, P. G., Power, M., and Silva, W. J. (2006). "Development of seismic ground-motion criteria for the ASCE/SEI 7 Standard." Paper 533, *Proceedings 8th U.S. National Conference on Earthquake Engineering*, April 18-22, 2006, San Francisco, California.

Dobry, R., Ramos, R., and Power, M. S. (1999). *Site Factors and Site Categories in Seismic Codes*, Technical Report MCEER-99-0010. Multidisciplinary Center for Earthquake Engineering Research.

Dobry, R., Borcherdt, R., Crouse, C. B., Idriss, I. M., Joyner, W. B., Martin, G. R., Power, M. S., Rinne, E. E., and Seed, R. B. (2000). "New site coefficients and site classifications system used in recent building seismic code provisions." *Earthquake Spectra*, 16(1), 41–67.

Federal Emergency Management Agency (FEMA). (2009). "NEHRP recommended seismic provisions for new buildings and other structures." *FEMA P-750*, Building Seismic Safety Council, Federal Emergency Management Agency.

Field, E. H. (2000). "A modified ground motion attenuation relationship for southern California that accounts for detailed site classification and a basin depth effect." *Bull. Seismological Soc. of Am.*, 90, S209–S221.

Harmsen, S. C. (1997). "Determination of site amplification in the Los Angeles urban area from inversion of strong motion records." *Bull. Seismological Soc. of Am.*, 87, 866–887.

Huang, Y.-N., Whittaker, A. S., and Luco, N. (2008). "Orientation of maximum spectral demand in the near-fault region." *Earthquake Spectra*, 24, 319–341.

Joyner, W. B., and Boore, D. M. (2000), "Recent developments in earthquake ground motion estimation." *Proceedings of the 6th International Conference on Seismic Zonation*, Palm Springs, Calif.

Luco, N., Ellingwood, B. R., Hamburger, R. O., Hooper, J. D., Kimball, J. K., and Kircher, C. A. (2007). "Risk-targeted versus current seismic design maps for the conterminous United States." *Proceedings, SEAOC 76th Annual Convention*, Structural Engineers Association of California, Sacramento, Calif.

Petersen, M. D., Frankel, A. D., Harmsen, S. C., Mueller, C. S., Haller, K. M., Wheeler, R. L., Wesson, R. L., Zeng, Y., Boyd, O. S., Perkins, D. M., Luco, N., Field, E. H., Wills, C. J., and Rukstales, K. S. (2008). "Documentation for the 2008 update of the United States National Seismic Hazard Maps." USGS Open File Report, U.S. Geological Survey, 1128.

Rodriguez-Marek, A., Bray, J. D., and Abrahamson, N. (2001). "An empirical geotechnical site response procedure." *Earthquake Spectra*, 17(1), 65–87.

Silva, W., Darragh, R., Gregor, N., Martin, G., Abrahamson, N., and Kircher, C. (2000). "Reassessment of site coefficients and near-fault factors for building code provisions." Program Element II, Report 98-HQ-GR-1010, to the U.S. Geological Survey.

Stewart, J. P., Liu, A. H., and Choi, Y. (2003). "Amplification factors for spectral acceleration in tectonically active regions." *Bull. Seismological Soc. of Am.*, 93(1), 332–352.

Steidl, J. H. (2000). "Site response in Southern California for probabilistic seismic hazard analysis." *Bull. Seismological Soc. of Am.*, 90, S149–S169.

OTHER REFERENCES

Abrahamson, N.A. (2000). "Effects of rupture directivity on probabilistic seismic hazard analysis." *Proceedings of the 6th International Conference on Seismic Zonation*, Palm Springs, California.

Somerville, P. G., Smith, N. F., Graves, R. W., and Abrahamson, N. A. (1997). "Modification of empirical strong ground motion attenuation relations to include the amplitude and duration effects of rupture directivity." *Seismological Res. Letters*, 68, 199–222.

CHAPTER C12
SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

C12.1 STRUCTURAL DESIGN BASIS

The performance expectations for structures designed in accordance with ASCE/SEI 7-10 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 maximum considered earthquake (MCE) 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 wind storm. 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).

Figure 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 that required by analysis. The SFRS is therefore expected to reach first significant yield for forces in excess of design forces. With increased lateral loading, additional plastic hinges form and the resistance increases at a reduced rate (following the solid curve) until the maximum

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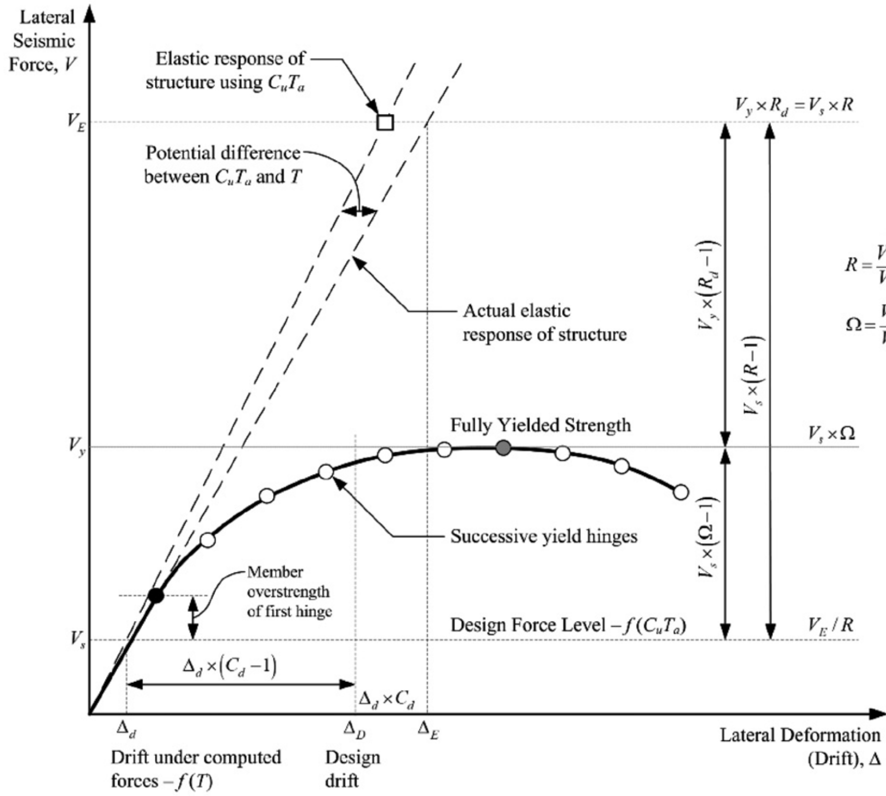


FIGURE C12.1-1 Inelastic Force-Deformation Curve

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 1984). 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.

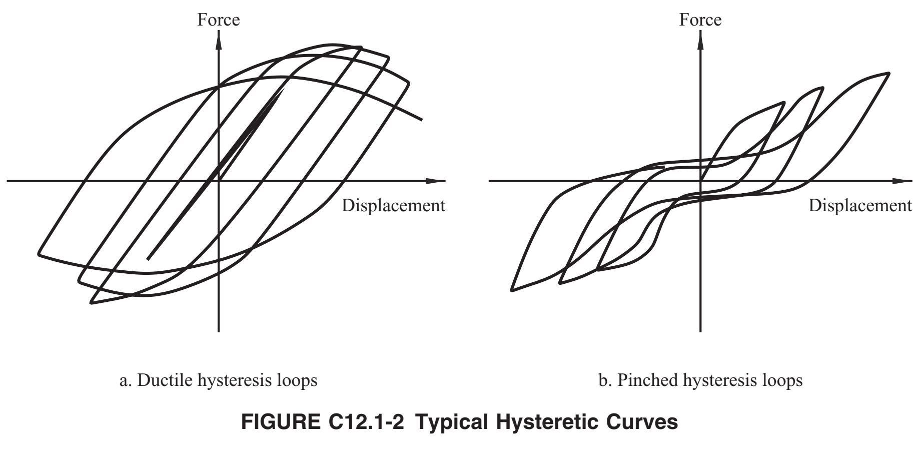
Figure C12.1-1 illustrates the significance of seismic design parameters contained in the standard, including the response modification coefficient, R; the deflection amplification factor, C_d; and the overstrength factor, Ω_0. The values of these parameters, provided in Table 12.2-1, as well as the 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 Figure C12.1-1 will not be able to form, the actual overstrength (Ω) will be 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 (Figure 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 Figure 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. Figure 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 Figure 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. While 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 do not possess as much toughness, lower values of R are specified.

The values for R, Ω_0, and C_d at the composite systems in Table 12.2-1 are similar to those for comparable systems of structural steel and reinforced concrete. Use of the tabulated values is allowed only when the design and detailing requirements in Section 14.3 are followed.

In a dual system, a three-dimensional space frame made up of columns and beams provides primary support for gravity loads. Primary lateral resistance is supplied by shear walls or braced frames, and secondary lateral resistance is provided by a moment frame complying with the requirements of Chapter 14.

Where a beam-column frame or slab-column frame lacks special detailing, it cannot act as an effective backup to a shear wall-subsystem, so there are no dual systems with ordinary moment frames. Instead, Table 12.2-1 permits the use of a shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls. Use of this defined system, which requires compliance with Section 12.2.5.8, offers a significant advantage over a simple combination of the two constituent ordinary reinforced concrete systems. Where those systems are simply combined, Section 12.2.3.3 would require use of seismic design parameters for an ordinary reinforced concrete moment frame.

In a cantilevered column system, stability of mass at the top is provided by one or more columns with base fixity acting as a single-degree-of-freedom system.

Cantilever column systems are essentially a special class of moment-resisting frame, except that they do not possess the redundancy and overstrength that most moment-resisting frames derive from sequential formation of yield or plastic hinges. Where a typical moment-resisting frame must form multiple plastic hinges in members to develop a yield mechanism, a cantilever column system develops hinges only at the base of the columns to form a mechanism. As a result, their overstrength is limited to that provided by material overstrength and by design conservatism.

It is permitted to construct cantilever column structures using any of the systems that can be used to develop moment frames, including ordinary and special steel; ordinary, intermediate, and special concrete; and timber frames. The system limitations for cantilever column systems reflect the type of moment frame detailing provided but with a limit on structural height, h_n, of 35 ft.

The value of R for cantilever column systems is derived from moment-resisting frame values where R is divided by Ω_0 but is not taken as less than 1 or greater than 2 1/2. This range accounts for the lack of sequential yielding in such systems. C_d is taken as equal to R recognizing that damping is quite low in these systems and inelastic displacement of these systems will not be less than the elastic displacement.

C12.2.2 Combinations of Framing Systems in Different Directions. Different seismic force-resisting systems can be used along each of the two orthogonal axes of the structure, as long as the respective values of R, Ω_0, and C_d are used. Depending on the combination selected, it is possible that one of the two systems may limit the extent of the overall system with regard to structural system limitations or structural height, h_n; the more restrictive of these would govern.

C12.2.3 Combinations of Framing Systems in the Same Direction. The intent of the provision requiring use of the most stringent seismic design parameters (R, Ω_0, and C_d) is to prevent mixed seismic force-resisting systems that could concentrate inelastic behavior in the lower stories.

C12.2.3.1 *R*, C_d, and Ω_0 Values for Vertical Combinations. This section expands upon Section 12.2.3 by specifying the requirements specific to the cases where (a) the value of R for the lower seismic force-resisting system is lower than that for the upper system, and (b) the value of R for the upper seismic force-resisting system is lower than that for the lower system.

The two cases are intended to account for all possibilities of vertical combinations of seismic force-resisting systems in the same direction. For a structure with a vertical combination of three or more seismic force-resisting systems in the same direction, Section 12.2.3.1 must be applied to the adjoining pairs of systems until the vertical combinations meet the requirements therein.

There are also exceptions to these requirements for conditions that do not affect the dynamic characteristics of the structure or that do not result in concentration of inelastic demand in critical areas.

C12.2.3.2 Two-Stage Analysis Procedure. A two-stage equivalent lateral force procedure is permitted where the lower portion of the structure has a minimum of 10 times the stiffness of the upper portion of the structure. In addition, the period of the entire structure is not permitted to be greater than 1.1 times the period of the upper portion considered as a separate structure supported at the transition from the upper to the lower portion. An example would be a concrete podium under a wood- or steel-framed upper portion of a structure. The upper portion may be analyzed for seismic forces and drifts using the values of R, Ω_0, and C_d for the upper portion as a separate structure. The seismic forces (e.g., shear and overturning) at the base of the upper portion are applied to the top of the lower portion and scaled up by the ratio of $(R/\rho)_{upper}$ to $(R/\rho)_{lower}$. The lower portion, which now includes the seismic forces from the upper portion, may then be analyzed using the values of R, Ω_0, and C_d for the lower portion of the structure.

C12.2.3.3 *R*, C_d, and Ω_0 Values for Horizontal Combinations. For almost all conditions, the least value of R of different seismic force-resisting systems in the same direction must be used in design. This requirement reflects the expectation that the entire system will undergo the same deformation with its behavior controlled by the least ductile system. However, for light-frame construction or flexible diaphragms meeting the listed conditions, the value of R for each independent line of resistance can be used. This exceptional condition is consistent with light-frame construction that uses the ground for parking with residential use above.

C12.2.4 Combination Framing Detailing Requirements. This requirement is provided so that the seismic force-resisting system with the highest value of R has the necessary ductile detailing throughout. The intent is that details common to both systems be designed to remain functional throughout the response to earthquake load effects to preserve the integrity of the seismic force-resisting system.

C12.2.5 System-Specific Requirements

C12.2.5.1 Dual System. The moment frame of a dual system must be capable of resisting at least 25% of the design seismic forces; this percentage is based on judgment. The purpose of the 25% frame is to provide a secondary seismic force-resisting

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system with higher degrees of redundancy and ductility to improve the ability of the building to support the service loads (or at least the effect of gravity loads) after strong earthquake shaking. The primary system (walls or bracing) acting together with the moment frame must be capable of resisting all of the design seismic forces. The following analyses are required for dual systems:

1. The moment frame and shear walls or braced frames must resist the design seismic forces considering fully the force and deformation interaction of the walls or braced frames and the moment frames as a single system. This analysis must be made in accordance with the principles of structural mechanics that consider the relative rigidities of the elements and torsion in the system. Deformations imposed upon members of the moment frame by their interaction with the shear walls or braced frames must be considered in this analysis.
2. The moment frame must be designed with sufficient strength to resist at least 25% of the design seismic forces.

C12.2.5.2 Cantilever Column Systems. Cantilever column systems are singled out for special consideration because of their unique characteristics. These structures often have limited redundancy and overstrength and concentrate inelastic behavior at their bases. As a result, they have substantially less energy dissipation capacity than other systems. A number of apartment buildings incorporating this system experienced very severe damage and, in some cases, collapsed in the 1994 Northridge (California) earthquake. Because the ductility of columns that have large axial stress is limited, cantilever column systems may not be used where individual column axial demands from seismic load effects exceed 15% of their available axial strength, including slenderness effects.

Elements providing restraint at the base of cantilever columns must be designed for seismic load effects, including overstrength, so that the strength of the cantilever columns is developed.

C12.2.5.3 Inverted Pendulum-Type Structures. Inverted pendulum-type structures do not have a unique entry in Table 12.2-1 because they can be formed from many structural systems. Inverted pendulum-type structures have more than half of their mass concentrated near the top (producing one degree of freedom in horizontal translation) and rotational compatibility of the mass with the column (producing vertical accelerations acting in opposite directions). Dynamic response amplifies this rotation; hence, the bending moment induced at the top of the column can exceed that computed using the procedures of Section 12.8. The requirement to design for a top moment that is one-half of the base moment calculated in accordance with Section 12.8 is based on analyses of inverted pendulums covering a wide range of practical conditions.

C12.2.5.4 Increased Structural Height Limit for Steel Eccentrically Braced Frames, Steel Special Concentrically Braced Frames, Steel Buckling-Restrained Braced Frames, Steel Special Plate Shear Walls, and Special Reinforced Concrete Shear Walls. The first criterion for an increased limit on structural height, h_n, precludes extreme torsional irregularity because premature failure of one of the shear walls or braced frames could lead to excessive inelastic torsional response. The second criterion, which is similar to the redundancy requirements, is to limit the structural height of systems that are too strongly dependent on any single line of shear walls or braced frames. The inherent torsion resulting from the distance between the center of mass and the center of rigidity must be included, but accidental torsional effects are neglected for ease of implementation.

C12.2.5.5 Special Moment Frames in Structures Assigned to Seismic Design Categories D through F. Special moment frames, either alone or as part of a dual system, are required to be used in Seismic Design Categories D through F where the structural height, h_n, exceeds 160 ft (or 240 ft for buildings that meet the provisions of Section 12.2.5.4) as indicated in Table 12.2-1. In shorter buildings where special moment frames are not required to be used, the special moment frames may be discontinued and supported on less ductile systems as long as the requirements of Section 12.2.3 for framing system combinations are followed.

For the situation where special moment frames are required, they should be continuous to the foundation. In cases where the foundation is located below the building's base, provisions for discontinuing the moment frames can be made as long as the seismic forces are properly accounted for and transferred to the supporting structure.

C12.2.5.6 Steel Ordinary Moment Frames. Steel ordinary moment frames (OMFs) are less ductile than steel special moment frames; consequently, their use is prohibited in structures assigned to Seismic Design Categories D, E, and F (see Table 12.2-1). Structures with steel OMFs, however, have exhibited acceptable behavior in past earthquakes where the structures were sufficiently limited in their structural height, number of stories, and seismic mass. The provisions in the standard reflect these observations. The exception is discussed separately below. Table C12.2.5.6-C12.2.5.7 summarizes the provisions.

C12.2.5.6.1 Seismic Design Category D or E. Single-story steel OMFs are permitted, provided that (a) the structural height, h_n, is a maximum of 65 ft (20 m), (b) the dead load supported by and tributary to the roof is a maximum of 20 lb/ft^2 (0.96 kN/m^2), and (c) the dead load of the exterior walls more than 35 ft (10.6 m) above the seismic base tributary to the moment frames is a maximum of 20 lb/ft^2 (0.96 kN/m^2).

In structures of light-frame construction, multistory steel OMFs are permitted, provided that (a) the structural height, h_n, is a maximum of 35 ft (10.6 m), (b) the dead load of the roof and each floor above the seismic base supported by and tributary to the moment frames are each a maximum of 35 lb/ft^2 (1.68 kN/m^2), and (c) the dead load of the exterior walls tributary to the moment frames is a maximum of 20 lb/ft^2 (0.96 kN/m^2).

Exception: Industrial structures, such as aircraft maintenance hangars and assembly buildings, with steel OMFs have performed well in past earthquakes with strong ground motions (EQE Inc. 1983, 1985, 1986a, 1986b, 1986c, and 1987); the exception permits single-story steel OMFs to be unlimited in height provided that (a) the structure is limited to the enclosure of equipment or machinery; (b) its occupants are limited to maintaining and monitoring the equipment, machinery, and their associated processes; (c) the sum of the dead load and equipment loads supported by and tributary to the roof is a maximum of 20 lb/ft^2 (0.96 kN/m^2); and (d) the dead load of the exterior wall system, including exterior columns more than 35 ft (10.6 m) above the seismic base is a maximum of 20 lb/ft^2 (0.96 kN/m^2). Though the latter two load limits (items c and d) are similar to those described in Section C12.2.5.6.1, there are meaningful differences.

The exception further recognizes that these facilities often require large equipment or machinery, and associated systems, not supported by or considered tributary to the roof, that support the intended operational functions of the structure, such as top

Table C12.2.5.6-C12.2.5.7 Summary of Conditions for OMFs and IMFs in Structures Assigned to Seismic Design Category D, E, or F (Refer to the Standard for Additional Requirements)

							Exterior Wall DL	
Section	Frame	SDC	Max. Number Stories	Light-Frame Construction	Max. h_n	Max. roof/floor DL (psf)	Max. (psf)	Wall[(1)] Height (ft)
12.2.5.6.1(a)	OMF	D, E	1	NA	65'0"	20	20	35'0"
12.2.5.6.1(a)-Exc	"	D, E	1	NA	NL	20	20	35'0"
12.2.5.6.1(b)	"	D, E	NL	Required	35'0"	35	20	0'0"
12.2.5.6.2	"	F	1	NA	65'0"	20	20	0'0"
12.2.5.7.1(a)	IMF	D	1	NA	65'0"	20	20	35'0"
12.2.5.7.1(a)-Exc	"	D	1	NA	NL	20	20	35'0"
12.2.5.7.1(b)	"	D	NL	NA	35'0"	NL	NL	NA
12.2.5.7.2(a)	"	E	1	NA	65'0"	20	20	35'0"
12.2.5.7.2(a)-Exc	"	E	1	NA	NL	20	20	35'0"
12.2.5.7.2(b)	"	E	NL	NA	35'0"	35	20	0'0"
12.2.5.7.3(a)	"	F	1	NA	65'0"	20	20	0'0"
12.2.5.7.3(b)	"	F	NL	Required	35'0"	35	20	0'0"

[1]Applies to portion of wall above listed wall height.
"NL" means "No Limit;" "NA" means "Not Applicable."

running bridge cranes, jib cranes, and liquid storage containment and distribution systems. To limit the seismic interaction between the seismic force-resisting systems and these components, the exception requires the weight of equipment or machinery that is not self-supporting (i.e., not freestanding) for all loads (e.g., dead, live, or seismic) to be included when determining compliance with the roof or exterior wall load limits. This *equivalent* equipment load shall be in addition to the loads listed above.

To determine the equivalent equipment load, the exception requires the weight to be considered fully (100%) tributary to an area not exceeding 600 ft^2 (55.8 m^2). This limiting area can be taken either to an adjacent exterior wall for cases where the weight is supported by an exterior column (which may also span to the first interior column) or to the adjacent roof for cases where the weight is supported entirely by an interior column or columns, but not both; nor can a fraction of the weight be allocated to each zone. Equipment loads within overlapping tributary areas should be combined in the same limiting area. Other provisions in the standard, as well as in past editions, require satisfying wall load limits tributary to the moment frame, but this requirement is not included in the exception in that it is based on a component-level approach that does not consider the interaction between systems in the structure. As such, the limiting area is considered to be a reasonable approximation of the tributary area of a moment frame segment for the purpose of this conversion. Although this weight allocation procedure may not represent an accurate physical distribution, it is considered to be a reasonable method for verifying compliance with the specified load limits to limit seismic interactions. The engineer must still be attentive to actual mass distributions when computing seismic loads. Further information is discussed in Section C11.1.3.

C12.2.5.6.2 Seismic Design Category F. Single-story steel OMFs are permitted, provided they meet. conditions (a) and (b) described in Section C12.2.5.6.1 for single-story frames and (c) the dead load of the exterior walls tributary to the moment frames is a maximum of 20 lb/ft^2 (0.96 kN/m^2).

C12.2.5.7 Steel Intermediate Moment Frames. Steel intermediate moment frames (IMFs) are more ductile than steel ordinary moment frames (OMFs) but less ductile than steel special moment frames; consequently, restrictions are placed on their use in structures assigned to Seismic Design Category D and their use is prohibited in structures assigned to Seismic Design Categories E and F (Table 12.2-1). As with steel OMFs, steel IMFs have also exhibited acceptable behavior in past earthquakes where the structures were sufficiently limited in their structural height, number of stories, and seismic mass. The provisions in the standard reflect these observations. The exceptions are discussed separately (below). Table C12.2.5.6-C12.2.5.7 summarizes the provisions.

C12.2.5.7.1 Seismic Design Category D. Single-story steel IMFs are permitted without limitations on dead load of the roof and exterior walls, provided the structural height, h_n, is a maximum of 35 ft (10.6 m). An increase to 65 ft (20 m) is permitted for h_n, provided that (a) the dead load supported by and tributary to the roof is a maximum of 20 lb/ft^2 (0.96 kN/m^2), and (b) the dead load of the exterior walls more than 35 ft (10.6 m) above the seismic base tributary to the moment frames is a maximum of 20 lb/ft^2 (0.96 kN/m^2).

The exception permits single-story steel IMFs to be unlimited in height, provided that they meet all of the conditions described in the exception to Section C12.2.5.6.1 for the same structures.

C12.2.5.7.2 Seismic Design Category E. Single-story steel IMFs are permitted, provided that they meet all of the conditions described in Section C12.2.5.6.1 for single-story OMFs.

The exception permits single-story steel IMFs to be unlimited in height, provided that they meet all of the conditions described in Section C12.2.5.6.1 for the same structures.

Multistory steel IMFs are permitted, provided that they meet all of the conditions described in Section C12.2.5.6.1 for multistory OMFs, except that the structure is not required to be of light-frame construction.

C12.2.5.7.3 Seismic Design Category F. Single-story steel IMFs are permitted, provided that (a) the structural height, h_n, is a maximum of 65 ft (20 m), (b) the dead load supported by and tributary to the roof is a maximum of 20 lb/ft^2 (0.96 kN/m^2), and (c) the dead load of the exterior walls tributary to the moment frames is a maximum of 20 lb/ft^2 (0.96 kN/m^2).

Multistory steel IMFs are permitted, provided that they meet all of the conditions described in the exception to Section C12.2.5.6.1 for multistory OMFs in structures of light-frame construction.

C12.2.5.8 Shear Wall-Frame Interactive Systems. For structures assigned to Seismic Design Category A or B (where seismic hazard is low), it is usual practice to design shear walls and frames of a shear wall-frame structure to resist lateral forces in proportion to their relative rigidities, considering interaction between the two subsystems at all levels. As discussed in Section C12.2.1, this typical approach would require use of a lower response modification coefficient, *R*, than that defined for shear wall-frame interactive systems. Where the special requirements of this section are satisfied, more reliable performance is expected, justifying a higher value of *R*.

C12.3 DIAPHRAGM FLEXIBILITY, CONFIGURATION IRREGULARITIES, AND REDUNDANCY

C12.3.1 Diaphragm Flexibility. Most seismic force-resisting systems have two distinct parts: the horizontal system that distributes lateral forces to the vertical elements and the vertical system that transmits lateral forces between the floor levels and the base of the structure.

The horizontal system may consist of diaphragms or a horizontal bracing system. For the majority of buildings, diaphragms offer the most economical and positive method of resisting and distributing seismic forces in the horizontal plane. Typically, diaphragms consist of a metal deck (with or without concrete), concrete slabs, and wood sheathing and/or decking. Although most diaphragms are flat, consisting of the floors of buildings, they also may be inclined, curved, warped, or folded configurations, and most diaphragms have openings.

The diaphragm stiffness relative to the stiffness of the supporting vertical seismic force-resisting system ranges from flexible to rigid and is important to define. Provisions defining diaphragm flexibility are given in Sections 12.3.1.1 through 12.3.1.3. If a diaphragm cannot be idealized as either flexible or rigid, explicit consideration of its stiffness must be included in the analysis.

The diaphragms in most buildings braced by wood light-frame shear walls are semirigid. Because semirigid diaphragm modeling is beyond the capability of available software for wood light-frame buildings, it is anticipated that this requirement will be met by evaluating force distribution using both rigid and flexible diaphragm models and taking the worse case of the two. Although this procedure is in conflict with common design practice, which typically includes only flexible diaphragm force distribution for wood light-frame buildings, it is one method of capturing the effect of the diaphragm stiffness.

C12.3.1.1 Flexible Diaphragm Condition. Section 12.3.1.1 defines broad categories of diaphragms that may be idealized as flexible, regardless of whether the diaphragm meets the calculated conditions of Section 12.3.1.3. These categories include the following:

a. Construction with relatively stiff vertical framing elements, such as steel-braced frames and concrete or masonry shear walls;
b. One-and two-family dwellings; and
c. Light-frame construction (e.g., construction consisting of light-frame walls and diaphragms) with or without nonstructural toppings of limited stiffness.

For item c above, compliance with story drift limits along each line of shear walls is intended as an indicator that the shear walls are substantial enough to share load on a tributary area basis and not require torsional force redistribution.

C12.3.1.2 Rigid Diaphragm Condition. Span-to-depth ratio limits are included in the deemed-to-comply condition as an indirect measure of the flexural contribution to diaphragm stiffness.

C12.3.1.3 Calculated Flexible Diaphragm Condition. A diaphragm is permitted to be idealized as flexible if the calculated diaphragm deflection (typically at midspan) between supports (lines of vertical elements) is greater than two times the average story drift of the vertical lateral force-resisting elements located at the supports of the diaphragm span.

C12.3.2 Irregular and Regular Classification. The configuration of a structure can significantly affect its performance during a strong earthquake that produces the ground motion contemplated in the standard. Structural configuration can be divided into two aspects: horizontal and vertical. Most seismic design provisions were derived for buildings that have regular configurations, but earthquakes have shown repeatedly that buildings that have irregular configurations suffer greater damage. This situation prevails even with good design and construction.

There are several reasons for the poor behavior of irregular structures. In a regular structure, the inelastic response, including energy dissipation and damage, produced by strong ground shaking tends to be well distributed throughout the structure. However, in irregular structures, inelastic behavior can be concentrated by irregularities and can result in rapid failure of structural elements in these areas. In addition, some irregularities introduce unanticipated demands into the structure, which designers frequently overlook when detailing the structural system. Finally, the elastic analysis methods typically used in the design of structures often cannot predict the distribution of earthquake demands in an irregular structure very well, leading to inadequate design in the areas associated with the irregularity. For these reasons, the standard encourages regular structural configurations and prohibits gross irregularity in buildings located on sites close to major active faults, where very strong ground motion and extreme inelastic demands are anticipated. The termination of seismic force-resisting elements at the foundation, however, is not considered to be a discontinuity.

C12.3.2.1 Horizontal Irregularity. A building may have a symmetric geometric shape without reentrant corners or wings but still be classified as irregular in plan because of its distribution of mass or vertical seismic force-resisting elements. Torsional effects in earthquakes can occur even where the centers of mass and rigidity coincide. For example, ground motion waves acting on a skew with respect to the building axis can cause torsion. Cracking or yielding in an asymmetric fashion also can cause torsion. These effects also can magnify the torsion caused by eccentricity between the centers of mass and rigidity. Torsional structural irregularities (Types 1a and 1b) are defined to address this concern.

A square or rectangular building with minor reentrant corners would still be considered regular, but large reentrant corners creating a crucifix form would produce an irregular structural configuration (Type 2). The response of the wings of this type of building generally differs from the response of the building as a whole, and this difference produces higher local forces than would be determined by application of the standard without modification. Other winged plan configurations (e.g., H-shapes) are classified as irregular even if they are symmetric because of the response of the wings.

Significant differences in stiffness between portions of a diaphragm at a level are classified as Type 3 structural irregularities because they may cause a change in the distribution of seismic

forces to the vertical components and may create torsional forces not accounted for in the distribution normally considered for a regular building.

Where there are discontinuities in the path of lateral force resistance, the structure cannot be considered regular. The most critical discontinuity defined is the out-of-plane offset of vertical elements of the seismic force-resisting system (Type 4). Such offsets impose vertical and lateral load effects on horizontal elements that are difficult to provide for adequately.

Where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system, the equivalent lateral force procedure of the standard cannot be applied appropriately, so the structure is considered to have an irregular structural configuration (Type 5).

Figure C12.3-1 illustrates horizontal structural irregularities.

C12.3.2.2 Vertical Irregularity. Vertical irregularities in structural configuration affect the responses at various levels and induce loads at these levels that differ significantly from the distribution assumed in the equivalent lateral force procedure given in Section 12.8.

A moment-resisting frame building might be classified as having a soft story irregularity (Type 1a or 1b) if one story is much taller than the adjoining stories and the design did not compensate for the resulting decrease in stiffness that normally would occur.

A building is classified as having a weight (mass) irregularity (Type 2) where the ratio of mass to stiffness in adjacent stories differs significantly. This difference typically occurs where a heavy mass (e.g., an interstitial mechanical floor) is placed at one level.

A vertical geometric irregularity (Type 3) applies regardless of whether the larger dimension is above or below the smaller one.

Vertical lateral force-resisting elements at adjoining stories that are offset from each other in the vertical plane of the elements and impose overturning demands on supporting structural elements, such as beams, columns, trusses, walls, or slabs, are classified as in-plane discontinuity irregularities (Type 4).

Buildings with a weak-story irregularity (Type 5a or 5b) tend to develop all of their inelastic behavior and consequent damage at the weak story, possibly leading to collapse.

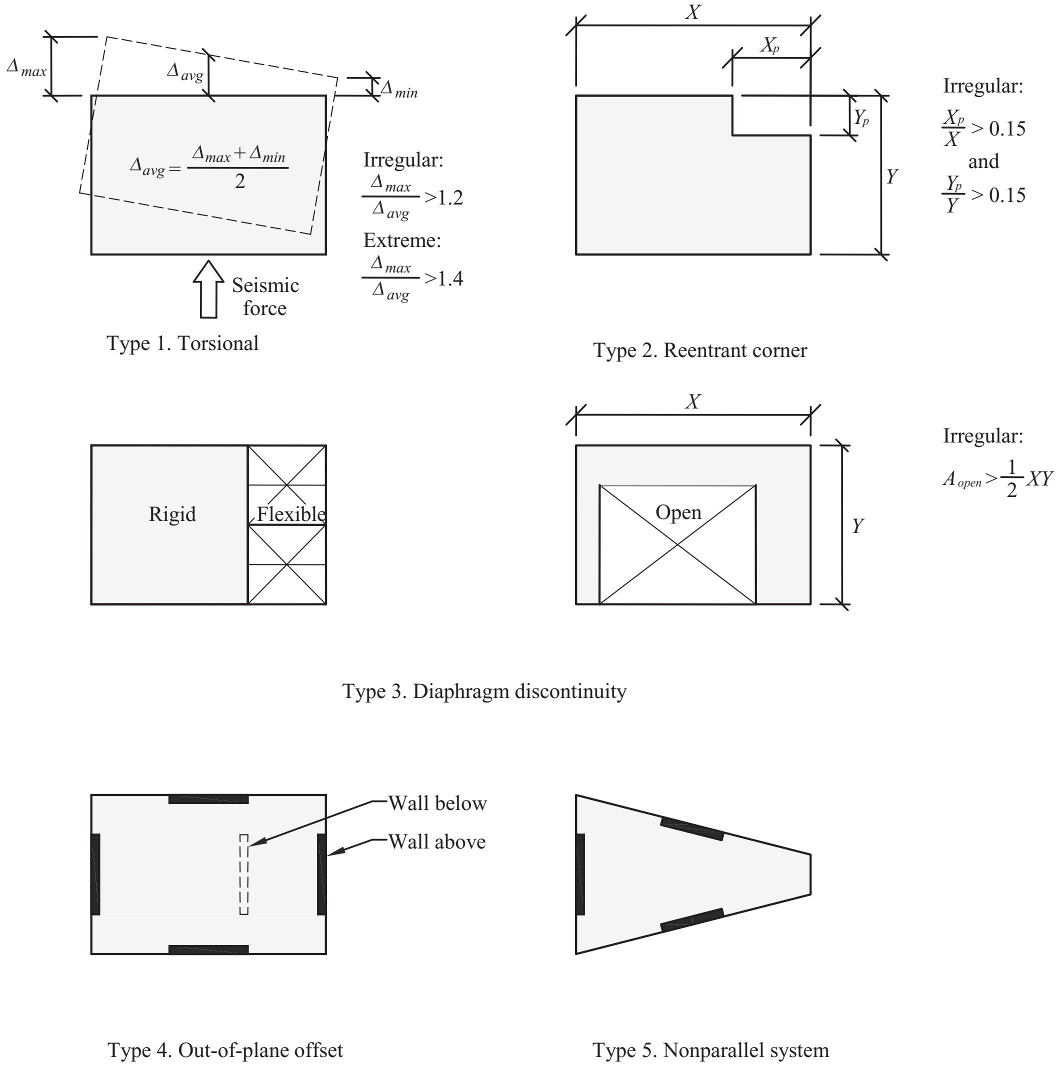

FIGURE C12.3-1 Horizontal Structural Irregularity Examples

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Figure C12.3-2 illustrates examples of vertical structural irregularities.

C12.3.3 Limitations and Additional Requirements for Systems with Structural Irregularities

C12.3.3.1 Prohibited Horizontal and Vertical Irregularities for Seismic Design Categories D through F. The prohibitions and limits caused by structural irregularities in this section stem from poor performance in past earthquakes and the potential to concentrate large inelastic demands in certain portions of the structure. Even where such irregularities are permitted, they should be avoided whenever possible in all structures.

C12.3.3.2 Extreme Weak Stories. Because extreme weak story irregularities are prohibited in Section 12.3.3.1 for buildings located in Seismic Design Categories D, E, and F, the limitations and exceptions in this section apply only to buildings assigned to Seismic Design Category B or C. Weak stories of structures assigned to Seismic Design Category B or C that are designed for seismic forces amplified by the overstrength factor, Ω_0, are exempted because reliable inelastic response is expected.

C12.3.3.3 Elements Supporting Discontinuous Walls or Frames. The purpose of requiring elements (e.g., beams, columns, trusses, slabs, and walls) that support discontinuous walls or frames to be designed to resist seismic load effects, including overstrength factor, is to protect the gravity load-carrying system against possible overloads caused by overstrength of the seismic force-resisting system. Either columns or beams may be subject to such failure; therefore, both should include this design requirement. Beams may be subject to failure caused by overloads in either the downward or upward directions of force. Examples include reinforced concrete beams, the weaker top laminations of glued laminated beams, or unbraced flanges of steel beams or trusses. Hence, the provision has not been limited simply to downward force, but instead to the larger context of "vertical load." Additionally, walls that support isolated point loads from frame columns or discontinuous perpendicular walls or walls with significant vertical offsets, as shown in Figures C12.3-3 and C12.3.3-4-1, can be subject to the same type of failure caused by overload.

The connection between the discontinuous element and the supporting member must be adequate to transmit the forces

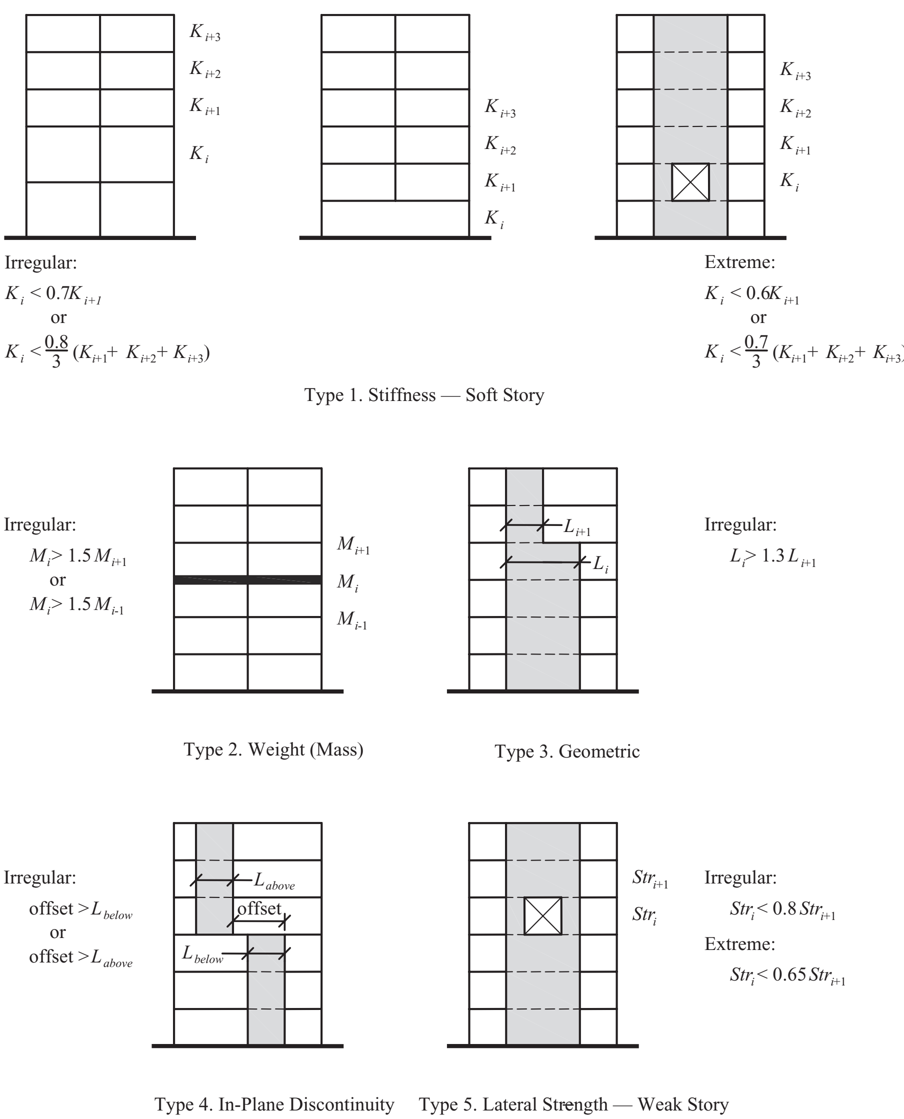


FIGURE C12.3-2 Vertical Structural Irregularities

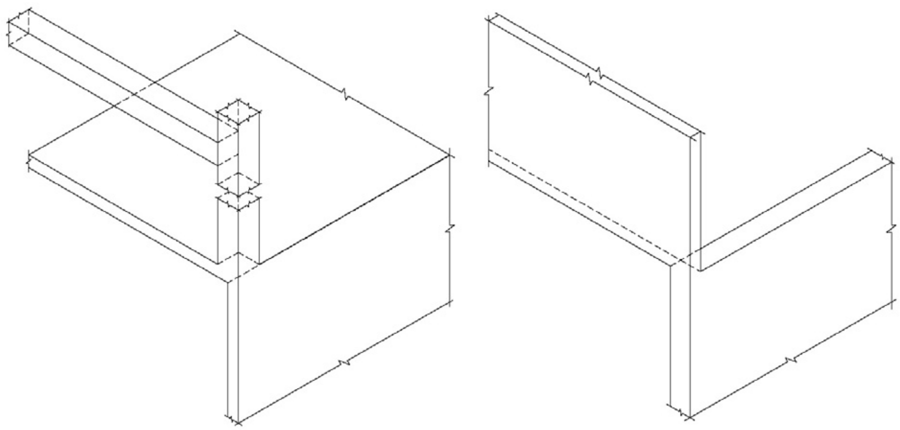


FIGURE C12.3-3 Vertical In-Plane-Discontinuity Irregularity from Columns or Perpendicular Walls (Type 4)

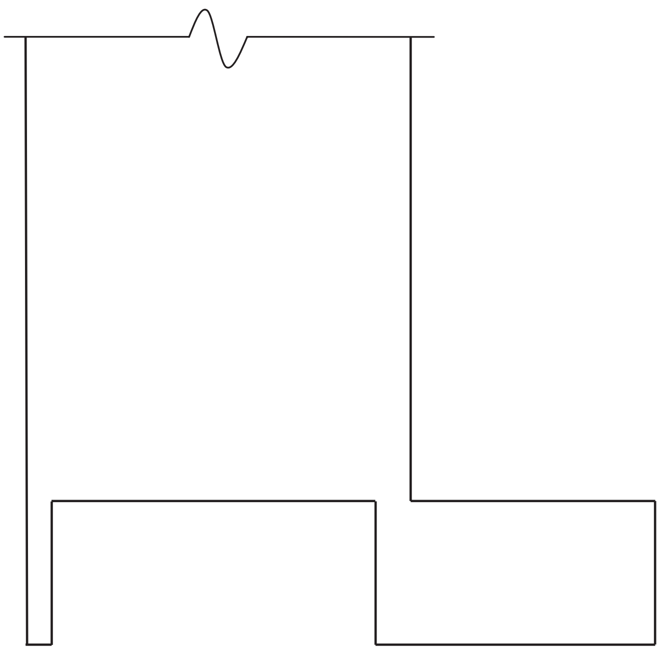


FIGURE C12.3-4 Vertical In-Plane-Discontinuity Irregularity from Walls with Significant Offsets (Type 4)

required for the design of the discontinuous element. For example, where the discontinuous element is required to comply with the seismic load effects, including overstrength factor in Section 12.4.3, as is the case for a steel column in a braced frame or a moment frame, its connection to the supporting member is required to be designed to transmit the same forces. These same seismic load effects are not required for shear walls, and, thus the connection between the shear wall and the supporting member would only need to be designed to transmit the loads associated with the shear wall.

For wood light-frame shear wall construction, the final sentence of Section 12.3.3.3 results in the shear and overturning connections at the base of a discontinued shear wall (i.e., shear fasteners and tie-downs) being designed using the load combinations of Section 2.3 or 2.4 rather than the load combinations with overstrength factor of Section 12.4.3 (Figure C12.3-5). The intent of the first sentence of Section 12.3.3.3 is to protect the system providing resistance to forces transferred from the shear wall by designing the system for amplified seismic load effects; strengthening of the shear wall anchorage to this system is not required to meet this intent.

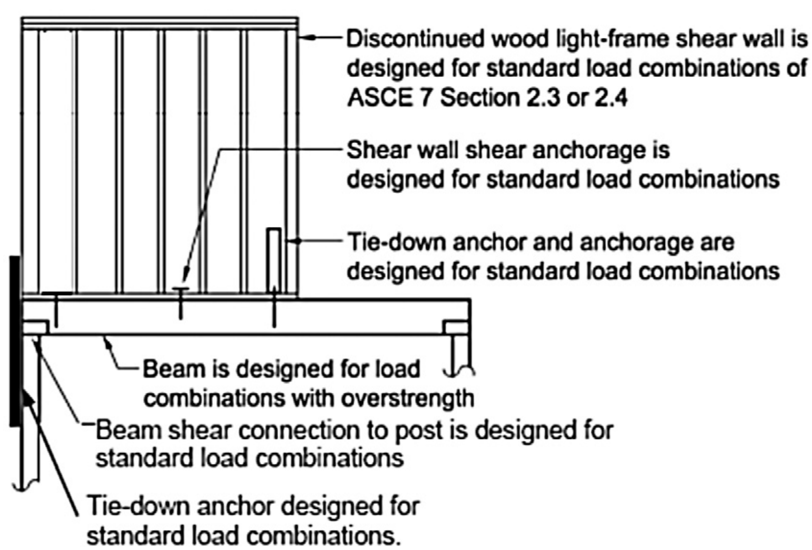


FIGURE C12.3-5 Discontinued Wood Light-Frame Shear Wall

C12.3.3.4 Increase in Forces Due to Irregularities for Seismic Design Categories D through F. The listed irregularities may result in loads that are distributed differently than assumed in the equivalent lateral force procedure of Section 12.8, especially as related to the interconnection of the diaphragm with vertical elements of the seismic force-resisting system. The 25% increase in force is intended to account for this difference. Where the force is calculated using the seismic load effects including overstrength factor, no further increase is warranted.

C12.3.4 Redundancy. The standard introduces a revised redundancy factor, ρ, for structures assigned to Seismic Design Category D, E, or F to quantify redundancy. The value of this factor is either 1.0 or 1.3. This factor has the effect of reducing the response modification coefficient, *R*, for less redundant structures, thereby increasing the seismic demand. The factor is specified in recognition of the need to address the issue of redundancy in the design.

The desirability of redundancy, or multiple lateral force-resisting load paths, has long been recognized. The redundancy provisions of this section reflect the belief that an excessive loss of story shear strength or development of an extreme torsional irregularity (Type 1b) may lead to structural failure. The value of ρ determined for each direction may differ.

C12.3.4.1 Conditions Where Value of ρ is 1.0. This section provides a convenient list of conditions where ρ is 1.0.

C12.3.4.2 Redundancy Factor, ρ, for Seismic Design Categories D through F. There are two approaches to establishing a redundancy factor, ρ, of 1.0. Where neither condition is satisfied, ρ is taken equal to 1.3. It is permitted to take ρ equal to 1.3 without checking either condition. A reduction in the value of ρ from 1.3 is not permitted for structures assigned to Seismic Design Category D that have an extreme torsional irregularity (Type 1b). Seismic Design Categories E and F are not also specified because extreme torsional irregularities are prohibited (see Section 12.3.3.1).

The first approach is a check of the elements outlined in Table 12.3-3 for cases where the seismic design story shear exceeds 35% of the base shear. Parametric studies (conducted by Building Seismic Safety Council Technical Subcommittee 2 but unpublished) were used to select the 35% value. Those studies indicated that stories with story shears of at least 35% of the base shear include all stories of low-rise buildings (buildings up to five to six stories) and about 87% of the stories of tall buildings. The intent of this limit is to exclude penthouses of most buildings and the uppermost stories of tall buildings from the redundancy requirements.

This approach requires the removal (or loss of moment resistance) of an individual lateral force-resisting element to determine its effect on the remaining structure. If the removal of elements, one by one, does not result in more than a 33% reduction in story strength or an extreme torsional irregularity, ρ may be taken as 1.0. For this evaluation, the determination of story strength requires an in-depth calculation. The intent of the check is to use a simple measure (elastic or plastic) to determine whether an individual member has a significant effect on the overall system. If the original structure has an extreme torsional irregularity to begin with, the resulting ρ is 1.3. Figure C12.3-6 presents a flowchart for implementing the redundancy requirements.

As indicated in Table 12.3-3, braced frame, moment frame, shear wall, and cantilever column systems must conform to redundancy requirements. Dual systems also are included but, in most cases, are inherently redundant. Shear walls or wall piers with a height-to-length aspect ratio greater than 1.0 within any story have been included; however, the required design of collector elements and their connections for Ω_0 times the design force may address the key issues. To satisfy the collector force requirements, a reasonable number of shear walls usually is required. Regardless, shear wall systems are addressed in this section so that either an adequate number of wall elements is included or the proper redundancy factor is applied. For wall piers, the height is taken as the height of the adjacent opening and generally is less than the story height.

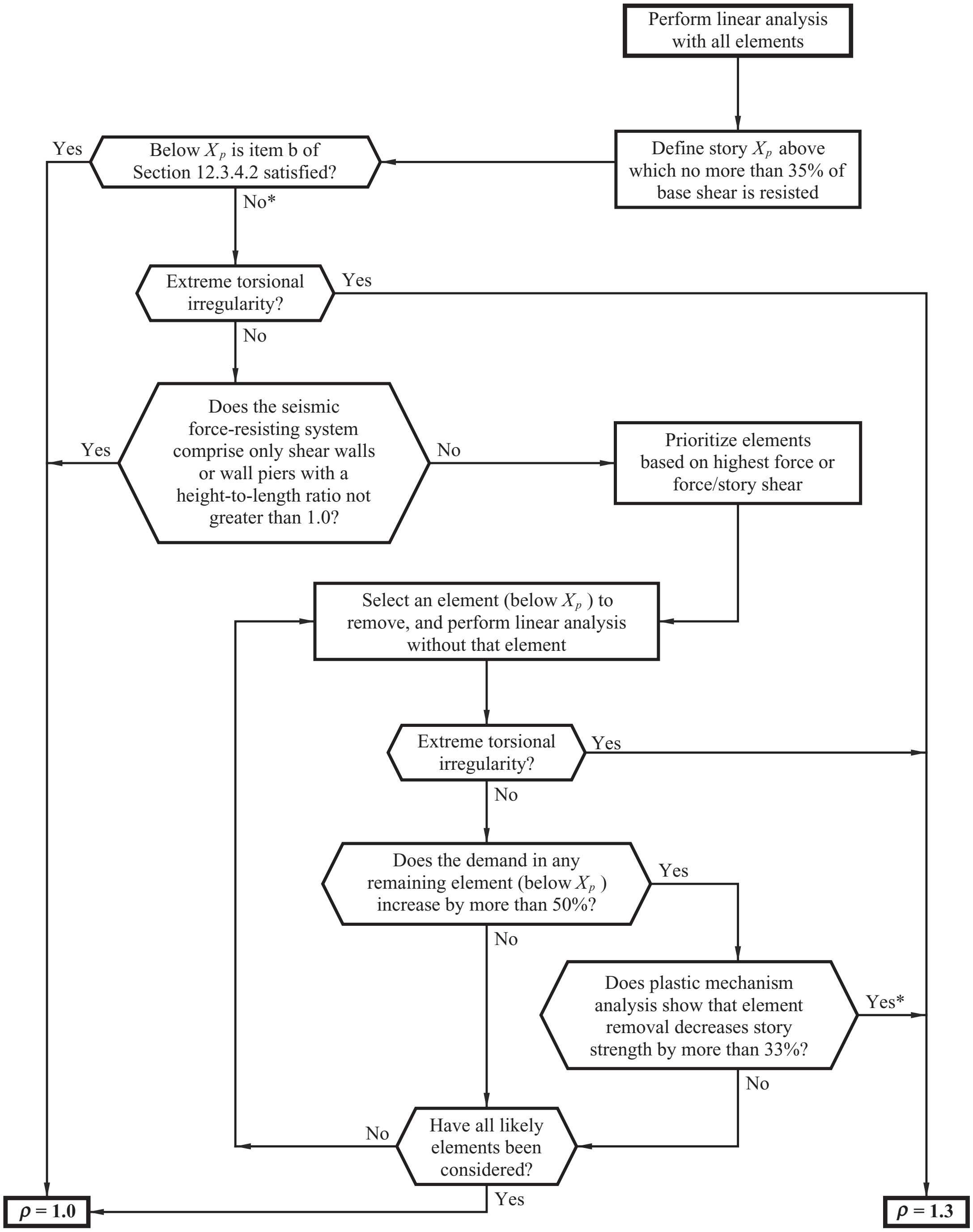
The second approach is a deemed-to-comply condition wherein the structure is regular and has a specified arrangement of seismic force-resisting elements to qualify for a ρ of 1.0. As part of the parametric study, simplified braced frame and moment frame systems were investigated to determine their sensitivity to the analytical redundancy criteria. This simple deemed-to-comply condition is consistent with the results of the study.

C12.4 SEISMIC LOAD EFFECTS AND COMBINATIONS

C12.4.1 Applicability. Structural elements designated by the engineer as part of the seismic force-resisting system typically are designed directly for seismic load effects. None of the seismic forces associated with the design base shear are formally assigned to structural elements that are not designated as part of the seismic force-resisting system, but such elements must be designed using the load conditions of Section 12.4 and must accommodate the deformations resulting from application of seismic loads.

C12.4.2 Seismic Load Effect. Section 12.4 presents the required combinations of seismic forces with other loads. The load combinations are taken from the basic load combinations of Chapter 2 of the standard with further elaboration of the seismic load effect, *E*. The seismic load effect includes horizontal and vertical components. The horizontal seismic load effects, E_h, are caused by the response of the structure to horizontal seismic ground motions, whereas the vertical seismic load effects are caused by the response of the structure to vertical seismic ground motions. The basic load combinations in Chapter 2 were duplicated and reformulated in Section 12.4 to clarify the intent of the provisions for the vertical seismic load effect term, E_v.

The concept of using an equivalent static load coefficient applied to the dead load to represent vertical seismic load effects was first introduced in ATC 3-06 (1978), where it was defined as simply +/− 0.2*D*. The load combinations where the vertical seismic load coefficient was to be applied assumed strength design load combinations. Neither ATC 3-06 (1978) nor the early versions of the NEHRP provisions (FEMA 2009a) clearly explained how the values of 0.2 were determined, but it is reasonable to assume that it was based on the judgment of the writers of those documents. It is accepted by the writers of this standard that vertical ground motions do occur and the value of +/–$0.2S_{DS}$ was determined based on consensus judgment. Many issues enter into the development of the vertical coefficient, including phasing of vertical ground motion and appropriate *R* factors, which make determination of a more precise value difficult. Although no specific rationale or logic is provided in editions of the NEHRP provisions (FEMA 2009a) on how the value of $0.2S_{DS}$ was determined, one possible way to rationalize the selection of the $0.2S_{DS}$ value is to recognize that it is equivalent to (2/3) (0.3) S_{DS}, where the 2/3 factor represents the often-assumed ratio between the vertical and horizontal components of motion, and the 0.3 factor represents the 30% in the



* or not considered

FIGURE C12.3-6 Calculation of the Redundancy Factor, ρ

100%-30% orthogonal load combination rule used for horizontal motions.

Although details regarding defining vertical ground motion spectra are currently well known, the committee elected not to define a vertical ground motion spectra in this standard because the approach provided by the equivalent static coefficient $0.2S_{DS}$ is adequate. For situations where one wishes to explicitly include the vertical component of ground motion in design analysis, one may use the vertical ground motion spectra definition that is provided in the "New Chapter 23, Vertical Ground Motions for Seismic Design" in the 2009 edition of the NEHRP provisions (FEMA 2009a).

C12.4.2.1 Horizontal Seismic Load Effect. Horizontal seismic load effects, E_h, are determined in accordance with Eq. 12.4-3 as $E_h = \rho Q_E$. Q_E is the seismic load effect of horizontal seismic forces from V or F_p. The purpose for E_h is to approximate the horizontal seismic load effect from the design basis earthquake to be used in load combinations including E, for the design of lateral force-resisting elements including diaphragms, vertical

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elements of seismic force-resisting systems as defined in Table 12.2-1, the design and anchorage of elements such as structural walls, and the design of nonstructural components.

C12.4.2.2 Vertical Seismic Load Effect. The vertical seismic load effect, E_v, is determined with Eq. 12.4-4 as $E_v = 0.2S_{DS}D$. E_v is permitted to be taken as zero in Eqs. 12.4-1, 12.4-2, 12.4-5, and 12.4-6 if S_{DS} is equal to or less than 0.125 and in Eq. 12.4-2 for determining demands on the soil-structure interface of foundations. E_v increases the load on beams and columns supporting seismic elements and increases the axial load in the P-M interaction of walls resisting seismic load effects.

C12.4.2.3 Seismic Load Combinations. The seismic load effect, E, is combined with the effects of other loads as set forth in Chapter 2 of the standard. For strength design, the load combinations in Section 2.3.2 that include E are modified in Section 12.4.2.3 to include the horizontal and vertical seismic load effects of Sections 12.4.2.1 and 12.4.2.2, respectively. Similarly, the basic load combinations for allowable stress design in Section 2.4.1 that include E are also modified in Section 12.4.2.3 to include the same seismic load effects.

For structures subject to the effects of flood loads or ice loads, Chapter 2 of the standard requires the consideration of additional load combinations and should be consulted to determine which combinations include E.

C12.4.3 Seismic Load Effect Including Overstrength Factor. Some elements of properly detailed structures are not capable of safely resisting ground-shaking demands through inelastic behavior. To ensure safety, these elements must be designed with sufficient strength to remain elastic. The overstrength factor, Ω_0, approximates the inherent overstrength in typical structures that have different seismic force-resisting systems.

C12.4.3.1 Horizontal Seismic Load Effect with Overstrength Factor. Horizontal seismic load effects with overstrength factor, E_{mh}, are determined in accordance with Eq. 12.4-7 as $E_{mh} = \Omega_o Q_E$. Q_E is the effect of horizontal seismic forces from V, F_{px}, or F_p. The purpose for E_{mh} is to approximate the maximum seismic load for the design of critical elements, including discontinuous systems, transfer beams and columns supporting discontinuous systems, and collectors.

Exception: Seismic load effects, E, multiplied by Ω_0, are an approximation of the maximum force these elements are ever likely to experience. The standard permits the seismic load effects, including overstrength factor, to be taken as less than the amount computed by applying Ω_0 to the design seismic forces where it can be determined that yielding of other elements in the structure limits the amount of load that can be delivered to the element and, therefore, the amount of force that can develop in the element.

As an example, the axial load in a column of a moment-resisting frame results from the shear forces in the beams that connect to this column. The axial loads caused by seismic load effects need never be taken as greater than the sum of the shears in these beams at the development of a full structural mechanism, considering the probable strength of the materials and strain-hardening effects. For frames controlled by beam hinge-type mechanisms, this load would typically be $2M_{pb}/L$ for steel frames where M_{pb} is the nominal plastic flexural strength of the beam as defined in AISC 341-10, and M_{pr}/l_n for concrete frames where M_{pr} is the probable flexural strength of the beam and l_n is the clear span length as defined in ACI 318-08.

In this context, the capacity of the element is its expected or median anticipated strength, considering potential variation in material yield strength and strain-hardening effects. When calculating the capacity of elements for this purpose, material strengths should not be reduced by strength reduction or resistance factors, ϕ.

C12.4.3.2 Load Combinations with Overstrength Factor. The seismic load effect including overstrength factor, E_m, is combined with other loads as set forth in Chapter 2 using the load combinations as set forth in Section 12.4.3.2. The purpose for load combinations with overstrength factor is to approximate the maximum seismic load combination for the design of critical elements including discontinuous systems, transfer beams and columns supporting discontinuous systems, and collectors.

C12.4.3.3 Allowable Stress Increase for Load Combinations with Overstrength. The allowable stress increase for load combinations with overstrength is to provide compatibility with past practice.

C12.4.4 Minimum Upward Force for Horizontal Cantilevers for Seismic Design Categories D through F. In Seismic Design Categories D, E, and F, horizontal cantilevers are designed for an upward force that results from an effective vertical acceleration of 1.2 times gravity. This design requirement is meant to provide some minimum strength in the upward direction and to account for possible dynamic amplification of vertical ground motions resulting from the vertical flexibility of the cantilever. The requirement is not applied to downward forces on cantilevers, for which the typical load combinations are used.

C12.5 DIRECTION OF LOADING

Seismic forces are delivered to a building through ground accelerations that may approach from any direction relative to the orthogonal directions of the building; therefore, seismic effects are expected to develop in both directions simultaneously. The standard requires structures to be designed for the most critical loading effects from seismic forces applied in any direction. The procedures outlined in this section are deemed to satisfy this requirement.

For horizontal structural elements such as beams and slabs, orthogonal effects may be minimal; however, design of vertical elements of the seismic force-resisting system that participate in both orthogonal directions is likely to be governed by these effects.

C12.5.1 Direction of Loading Criteria. For structures with orthogonal seismic force-resisting systems, the most critical load effects can typically be computed using a pair of orthogonal directions that coincide with the principal axes of the structure. Structures with nonparallel or nonorthogonal systems may require a set of orthogonal direction pairs to determine the most critical load effects. If a three-dimensional mathematical model is used, the analyst must be attentive to the orientation of the global axes in the model in relation to the principal axes of the structure.

C12.5.2 Seismic Design Category B. Recognizing that design of structures assigned to Seismic Design Category (SDC) B is often controlled by nonseismic load effects and, therefore, is not sensitive to orthogonal loadings regardless of any horizontal structural irregularities, it is permitted to determine the most critical load effects by considering that the maximum response can occur in any single direction; simultaneous application of response in the orthogonal direction is not required. Typically, the two directions used for analysis coincide with the principle axes of the structure.

C12.5.3 Seismic Design Category C. Design of structures assigned to SDC C often parallels the design of structures assigned to SDC B and, therefore, as a minimum conforms to Section 12.5.2. Although it is not likely that design of the seismic force-resisting systems in regular structures assigned to SDC C would be sensitive to orthogonal loadings, special consideration must be given to structures with nonparallel or nonorthogonal systems (Type 5 horizontal structural irregularity) to avoid overstressing by different directional loadings. In this case, the standard provides two methods to approximate simultaneous orthogonal loadings and requires a three-dimensional mathematical model of the structure for analysis in accordance with Section 12.7.3.

The orthogonal combination procedure in item (a) of Section 12.5.3 combines the effects from 100% of the seismic load applied in one direction with 30% of the seismic load applied in the perpendicular direction. This general approximation—the "30% rule"—was introduced by Rosenblueth and Contreras (1977) based on earlier work by A. S. Veletsos and also N. M. Newmark (cited in Rosenblueth and Contreras 1977), as an alternative to performing the more rational, yet computationally demanding, response history analysis, and is applicable to any elastic structure. Combining effects for seismic loads in each direction, and accidental torsion in accordance with Sections 12.8.4.2 and 12.8.4.3, results in the following 16 load combinations:

- Q_E = +/− Q_{E_X+AT} +/− $0.3Q_{E\_Y}$ where Q_{E_Y} = effect of Y-direction load at the center of mass (Section 12.8.4.2);
- Q_E = +/− Q_{E_X-AT} +/− $0.3Q_{E\_Y}$ where Q_{E_X} = effect of X-direction load at the center of mass (Section 12.8.4.2);
- Q_E = +/− Q_{E_Y+AT} +/− $0.3Q_{E\_X}$. where AT = accidental torsion computed in accordance with Sections 12.8.4.2 and 12.8.4.3; and
- Q_E = +/− Q_{E_Y-AT} +/− $0.3Q_{E\_X}$.

Though the standard permits combining effects from forces applied independently in any pair of orthogonal directions (to approximate the effects of concurrent loading), accidental torsion need not be considered in the direction that produces the lesser effect, per Section 12.8.4.2. This provision is sometimes disregarded when using a mathematical model for three-dimensional analysis that can automatically include accidental torsion, which then results in 32 load combinations.

The maximum effect of seismic forces, Q_E, from orthogonal load combinations is modified by the redundancy factor, ρ, or the overstrength factor, Ω_0, where required, and the effects of vertical seismic forces, E_v, are considered in accordance with Section 12.4, to obtain the seismic load effect, E.

These orthogonal combinations should not be confused with uniaxial modal combination rules, such as the square root of the sum of the squares (SRSS) or complete quadratic combination (CQC) method. In past standards, an acceptable alternative to the above was to use the SRSS method to combine effects of the two orthogonal directions, where each term computed is assigned the sign that resulted in the most conservative result. This method is no longer in common use. Although both approaches described for considering orthogonal effects are approximations, it is important to note that they were developed with consideration of results for a square building.

Orthogonal effects can alternatively be considered by performing three-dimensional response history analyses (see Chapter 16) with application of orthogonal ground motion pairs applied simultaneously in any two orthogonal directions. If the structure is located within 3 mi (5 km) of an active fault, the ground motion pair should be rotated to the fault-normal and fault-parallel directions of the causative fault.

C12.5.4 Seismic Design Categories D through F. The direction of loading for structures assigned to SDCs D, E, or F conforms to Section 12.5.3 for structures assigned to SDC C. If a Type 5 horizontal structural irregularity exists, then orthogonal effects are similarly included in design. Recognizing the higher seismic risk associated with structures assigned to SDCs D, E, or F, the standard provides additional requirements for vertical members coupled between intersecting seismic force-resisting systems.

C12.6 ANALYSIS PROCEDURE SELECTION

Table 12.6-1 provides the permitted analysis procedures for all seismic design categories. The table is applicable only to buildings without seismic isolation (Chapter 17) or passive energy devices (Chapter 18) for which there are additional requirements in Sections 17.4 and 18.2.4, respectively.

The four basic procedures provided in Table 12.6-1 are the equivalent lateral force (ELF, Section 12.8), the modal response spectrum (MRS, Section 12.9), the linear response history (LRH, Section 16.1), and the nonlinear response history (NRH, Section 16.2) analysis procedures. Nonlinear static pushover analysis is not provided as an "approved" analysis procedure in the standard.

The ELF procedure is allowed for all buildings assigned to Seismic Design Category B or C and for all buildings assigned to Seismic Design Category D, E, or F, except for the following:

- Structures with structural height, h_n, > 160 ft (48.8 m) and $T > 3.5T_s$;
- Structures with structural height, h_n, > 160 ft (48.8 m) and $T \leq 3.5T_s$ but with one or more of the structural irregularities in Table 12.3-1 or 12.3-2; and
- Structures with structural height, h_n, < 160 ft (48.8 m) and with one or more of the following structural irregularities: torsion or extreme torsion (Table 12.3-1); or soft story, extreme soft story, weight (mass), or vertical geometric (Table 12.3-2).

$T_s = S_{D1}/S_{DS}$ is the period at which the horizontal and descending parts of the design response spectrum intersect (Fig. 11-4.1). The

Table C12.6-1 Values of $3.5T_s$ for Various Cities and Site Classes

			$3.5T_s$ (s) for Site Class			
Location	S_s (g)	S_1 (g)	**A & B**	**C**	**D**	**E**
Denver	0.219	0.057	0.91	1.29	1.37	1.07
Boston	0.275	0.067	0.85	1.21	1.30	1.03
New York City	0.359	0.070	0.68	0.97	1.08	0.93
Las Vegas	0.582	0.179	1.08	1.50	1.68	1.89
St. Louis	0.590	0.169	1.00	1.40	1.60	1.81
San Diego	1.128	0.479	1.31	1.73	1.99	2.91
Memphis	1.341	0.368	0.96	1.38	1.59	2.25
Charleston	1.414	0.348	0.86	1.25	1.47	2.08
Seattle	1.448	0.489	1.18	1.55	1.78	2.63
San Jose	1.500	0.600	1.40	1.82	2.10	2.12
Salt Lake City	1.672	0.665	1.39	1.81	2.09	3.10

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value of T_s depends on the site class because S_{DS} and S_{D1} include such effects. Where the ELF procedure is not allowed, the analysis must be performed using modal response spectrum or response history analysis.

The use of the ELF procedure is limited to buildings with the listed structural irregularities because the procedure is based on an assumption of a gradually varying distribution of mass and stiffness along the height and negligible torsional response. The basis for the $3.5T_s$ limitation is that the higher modes become more dominant in taller buildings (Lopez and Cruz 1996, Chopra 2007), and as a result, the ELF procedure may underestimate the seismic base shear and may not correctly predict the vertical distribution of seismic forces in taller buildings.

As Table C12.6-1 demonstrates, the value of $3.5T_s$ generally increases as ground motion intensity increases and as soils become softer. Assuming a fundamental period of approximately 0.1 times the number of stories, the maximum structural height, h_n, for which the ELF procedure applies ranges from about 10 stories for low seismic hazard sites with firm soil to 30 stories for high seismic hazard sites with soft soil. Because this trend was not intended, the 160-ft (48.8-m) height limit is introduced.

C12.7 MODELING CRITERIA

C12.7.1 Foundation Modeling. Structural systems consist of three interacting subsystems: the structural framing (girders, columns, walls, and diaphragms), the foundation (footings, piles, and caissons), and the supporting soil. The ground motion that a structure experiences, as well as the response to that ground motion, depends on the complex interaction among these subsystems.

Those aspects of ground motion that are affected by site characteristics are assumed to be independent of the structure-foundation system because these effects would occur in the free field in the absence of the structure. Hence, site effects are considered separately (Sections 11.4.2 through 11.4.4 and Chapters 20 and 21).

Given a site-specific ground motion or response spectrum, the dynamic response of the structure depends on the foundation system and on the characteristics of the soil that support the system. The dependence of the response on the structure-foundation-soil system is referred to as soil-structure interaction. Such interactions usually, but not always, result in a reduction of seismic base shear. This reduction is caused by the flexibility of the foundation-soil system and an associated lengthening of the fundamental period of vibration of the structure. In addition, the soil system may provide an additional source of damping. However, that total displacement typically increases with soil-structure interaction.

If the foundation is considered to be rigid, the computed base shears are usually conservative, and it is for this reason that rigid foundation analysis is permitted. The designer may neglect soil-structure interaction or may consider it explicitly in accordance with Section 12.13.3 or implicitly in accordance with Chapter 19.

As an example, consider a moment-frame building without a basement and with moment-frame columns supported on footings designed to support shear and axial loads, i.e., pinned column bases. If foundation flexibility is not considered, the columns should be restrained horizontally and vertically, but not rotationally. Consider a moment-frame building with a basement. For this building, horizontal restraint may be provided at the level closest to grade, as long as the diaphragm is designed to transfer the shear out of the moment frame. Because the columns extend through the basement, they may also be restrained rotationally and vertically at this level. However, it is often preferable to extend the model through the basement and provide the vertical and rotational restraints at the foundation elements, which is more consistent with the actual building geometry.

C12.7.2 Effective Seismic Weight. During an earthquake, the structure accelerates laterally, and these accelerations of the structural mass produce inertial forces. These inertial forces, accumulated over the height of the structure, produce the seismic base shear.

When a building vibrates during an earthquake, only that portion of the mass or weight that is physically tied to the structure needs to be considered as effective. Hence, live loads (e.g., loose furniture, loose equipment, and human occupants) need not be included. However, certain types of live loads, such as storage loads, may develop inertial forces, particularly where they are densely packed.

Also considered as contributing to effective seismic weight are the following:

1. All permanent equipment (e.g., air conditioners, elevator equipment, and mechanical systems);
2. Partitions to be erected or rearranged as specified in Section 4.3.2 (greater of actual partition weight and 10 lb/ft^2 of floor area);
3. 20% of significant snow load ($p_f > 30$ lb/ft^2); and
4. The weight of landscaping and similar materials.

The full snow load need not be considered because maximum snow load and maximum earthquake load are unlikely to occur simultaneously and loose snow does not move with the roof.

C12.7.3 Structural Modeling. The development of a mathematical model of a structure is always required because the story drifts and the design forces in the structural members cannot be determined without such a model. In some cases, the mathematical model can be as simple as a free-body diagram as long as the model can appropriately capture the strength and stiffness of the structure.

The most realistic analytical model is three-dimensional, includes all sources of stiffness in the structure and the soil-foundation system as well as P-delta effects, and allows for nonlinear inelastic behavior in all parts of the structure-foundation soil system. Development of such an analytical model is very time consuming, and such analysis is rarely warranted for typical building designs performed in accordance with the standard. Instead of performing a nonlinear analysis, inelastic effects are accounted for indirectly in the linear analysis methods by means of the response modification coefficient, R, and the deflection amplification factor, C_d.

Using modern software, it often is more difficult to decompose a structure into planar models than it is to develop a full three-dimensional model, so three-dimensional models are now commonplace. Increased computational efficiency also allows efficient modeling of diaphragm flexibility. Three-dimensional models are required where the structure has horizontal torsional (Type 1), out-of-plane offset (Type 4), or nonparallel system (Type 5) irregularities.

Analysis using a three-dimensional model is not required for structures with flexible diaphragms that have horizontal out-of-plane offset irregularities. It is not required because the irregularity imposes seismic load effects in a direction other than the direction under consideration (orthogonal effects) because of eccentricity in the vertical load path caused by horizontal offsets of the vertical lateral force-resisting elements from story to story.

This situation is not likely to occur, however, with flexible diaphragms to an extent that warrants such modeling. The eccentricity in the vertical load path causes a redistribution of seismic design forces from the vertical elements in the story above to the vertical elements in the story below in essentially the same direction. The effect on the vertical elements in the orthogonal direction in the story below is minimal. Three-dimensional modeling may still be required for structures with flexible diaphragms caused by other types of horizontal irregularities (e.g., nonparallel system).

In general, the same three-dimensional model may be used for the equivalent lateral force, the modal response spectrum, and the linear response history analysis procedures. Modal response spectrum and linear response history analyses require a realistic modeling of structural mass; the response history method also requires an explicit representation of inherent damping. Five percent of critical damping is automatically included in the modal response spectrum approach. Chapter 16 and the related commentary have additional information on linear and nonlinear response history analysis procedures.

It is well known that deformations in the panel zones of the beam-column joints of steel moment frames are a significant source of flexibility. Two different mechanical models for including such deformations are summarized in Charney and Marshall (2006). These methods apply to both elastic and inelastic systems. For elastic structures, centerline analysis provides reasonable, but not always conservative, estimates of frame flexibility. Fully rigid end zones should not be used because this will always result in an overestimation of lateral stiffness in steel moment-resisting frames. Partially rigid end zones may be justified in certain cases, such as where doubler plates are used to reinforce the panel zone.

Including the effect of composite slabs in the stiffness of beams and girders may be warranted in some circumstances. Where composite behavior is included, due consideration should be paid to the reduction in effective composite stiffness for portions of the slab in tension (Schaffhausen and Wegmuller 1977 and Liew et al. 2001).

For reinforced concrete buildings, it is important to address the effects of axial, flexural, and shear cracking in modeling the effective stiffness of the structural elements. Determining appropriate effective stiffness of the structural elements should take into consideration the anticipated demands on the elements, their geometry, and the complexity of the model. Recommendations for computing cracked section properties may be found in Paulay and Priestley (1992) and similar texts.

C12.7.4 Interaction Effects. The interaction requirements are intended to prevent unexpected failures in members of moment-resisting frames. Figure C12.7-1 illustrates a typical situation where masonry infill is used, and this masonry is fitted tightly against reinforced concrete columns. Because the masonry is much stiffer than the columns, hinges in a column form at the top of the column and at the top of the masonry rather than at the top and bottom of the column. If the column flexural capacity is M_p, the shear in the columns increases by the factor H/h, and this increase may cause an unexpected nonductile shear failure in the columns. Many building collapses have been attributed to this effect.

C12.8 EQUIVALENT LATERAL FORCE PROCEDURE

The equivalent lateral force (ELF) procedure provides a simple way to incorporate the effects of inelastic dynamic response into a linear static analysis. This procedure is useful in preliminary design of all structures and is allowed for final design of the vast majority of structures. The procedure is valid only for structures without significant discontinuities in mass and stiffness along the height, where the dominant response to ground motions is in the horizontal direction without significant torsion.

The ELF procedure has three basic steps:

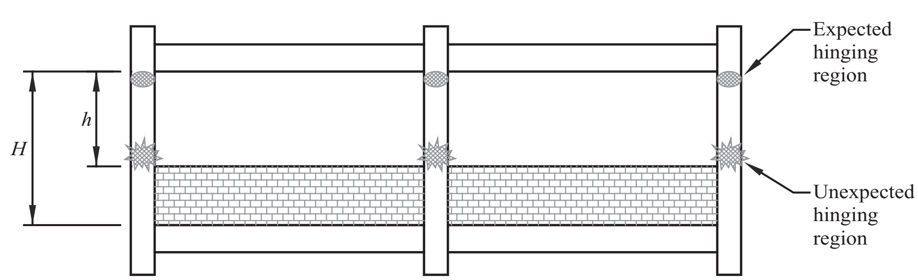
1. Determine the seismic base shear, V;
2. Distribute V vertically along the height of the structure; and
3. Distribute V horizontally across the width and breadth of the structure.

Each of these steps is based on a number of simplifying assumptions. A broader understanding of these assumptions may be obtained from any structural dynamics textbook that emphasizes seismic applications.

C12.8.1 Seismic Base Shear. Treating the structure as a single-degree-of-freedom system with 100% mass participation in the fundamental mode, Eq. 12.8-1 simply expresses V as the product of the effective seismic weight, W, and the seismic response coefficient, C_s, which is a period-dependent, spectral pseudoacceleration, in g units. C_s is modified by the response modification coefficient, R, and the importance factor, I_e, as appropriate, to account for inelastic behavior and to provide for improved performance for high-occupancy or essential structures.

C12.8.1.1 Calculation of Seismic Response Coefficient. The standard prescribes five equations for determining C_s. Eqs. 12.8-2, 12.8-3, and 12.8-4 are illustrated in Figure C12.8-1.

Equation 12.8-2 controls where $0.0 < T < T_s$ and represents the constant acceleration part of the design response spectrum (Section 11.4.5). In this region, C_s is independent of period.



Expected plastic hinge capacity = M_p

Expected column shear = $2M_p/H$

Actual column shear = $2M_p/h$

FIGURE C12.7-1 Undesired Interaction Effects

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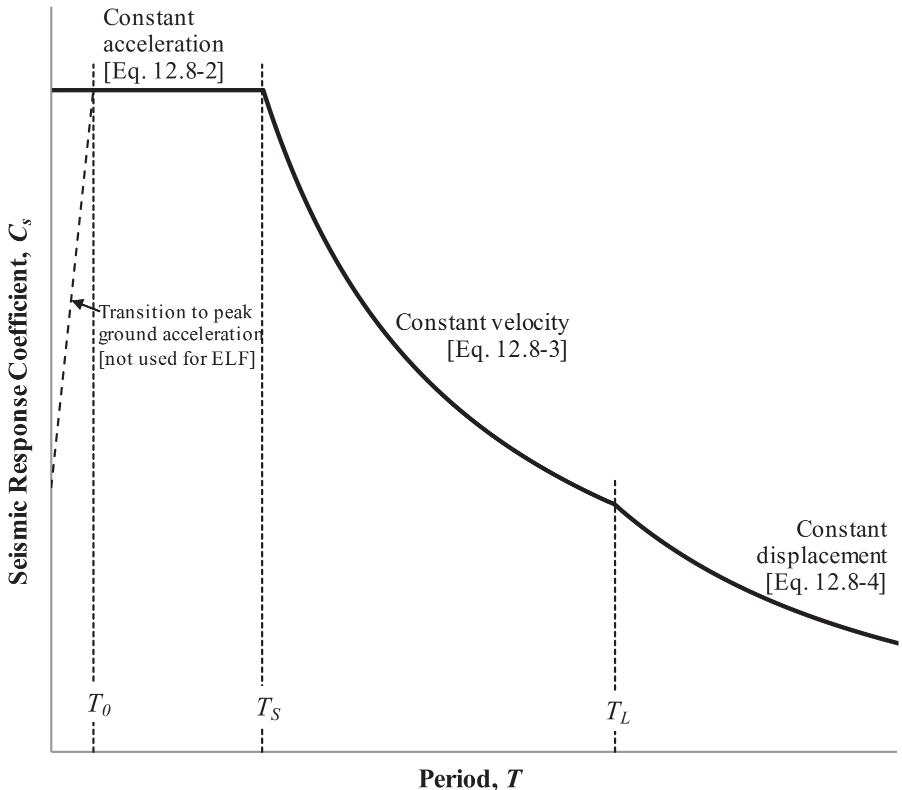

FIGURE C12.8-1 Seismic Response Coefficient Versus Period

Although the theoretical design response spectrum shown in Fig. 11.4-1 illustrates a transition in pseudoacceleration to the peak ground acceleration as the fundamental period, T, approaches zero from T_0, this transition is not used in the ELF procedure. One reason is that simple reduction of the response spectrum by $(1/R)$ in the short period region would exaggerate inelastic effects.

Equation 12.8-3, representing the constant velocity part of the spectrum, controls where $T_s < T < T_L$. In this region, the seismic response coefficient is inversely proportional to period, and the pseudovelocity (pseudoacceleration divided by circular frequency, ω, assuming steady-state response) is constant. T_L, the long-period transition period, represents the transition to constant displacement and is provided in Figs. 22-12 through 22-16. T_L ranges from 4 s in the north-central conterminous states and western Hawaii to 16 s in the Pacific Northwest and in western Alaska.

Equation 12.8-4, representing the constant displacement part of the spectrum, controls where $T > T_L$. Given the current mapped values of T_L, this equation only affects long-period structures. The transition period has recently received increased attention because displacement response spectra from the 2010 magnitude 8.8 Chilean earthquake indicate that a considerably lower transition period is possible in locations controlled by subduction zone earthquakes.

The final two equations repr esent minimum base shear levels for design. Equation 12.8-5 is the minimum base shear and primarily affects sites in the far field. This equation provides an allowable strength of approximately 3% of the weight of the structure. This minimum base shear was originally enacted in 1933 by the state of California (Riley Act). Based on research conducted in the ATC-63 project (FEMA 2009b), it was determined that this equation provides an adequate level of collapse resistance for long-period structures when used in conjunction with other provisions of the standard.

Equation 12.8-6 applies to sites near major active faults (as reflected by values of S_1) where pulse-type effects can increase long-period demands.

C12.8.1.2 Soil Structure Interaction Reduction. Soil-structure interaction, which can significantly influence the dynamic response of a structure during an earthquake, is addressed in Chapter 19.

C12.8.1.3 Maximum S_S Value in Determination of C_s. The maximum value of S_S was created as hazard maps were revised in 1997. The cap on S_S reflects engineering judgment about performance of code-complying buildings in past earthquakes, so the structural height, period, and regularity conditions required for use of the limit are important qualifiers.

C12.8.2 Period Determination. The fundamental period, T, for an elastic structure is used to determine the design base shear, V, as well as the exponent, k, that establishes the distribution of V along the height of the structure (see Section 12.8.3). T may be computed using a mathematical model of the structure that incorporates the requirements of Section 12.7 in a properly substantiated analysis. Generally, this type of analysis is performed using a computer program that incorporates all deformational effects (e.g., flexural, shear, and axial) and accounts for the effect of gravity load on the stiffness of the structure. For many structures, however, the sizes of the primary structural members are not known at the outset of design. For preliminary design, as well as instances where a substantiated analysis is not used, the standard provides formulas to compute an approximate fundamental period, T_a (see Section 12.8.2.1). These periods represent lower-bound estimates of T for different structure types. Period

determination is typically computed for a mathematical model that is fixed at the base. That is, the base where seismic effects are imparted into the structure is globally restrained (e.g., horizontally, vertically, and rotationally). Column base modeling (i.e., pinned or fixed) for frame-type seismic force-resisting systems is a function of frame mechanics, detailing, and foundation (soil) rigidity; attention should be given to the adopted assumption. However, this conceptual restraint is not the same for the structure as is stated above. Soil flexibility may be considered for computing T (typically assuming a rigid foundation element). The engineer should be attentive to the equivalent linear soil-spring stiffness used to represent the deformational characteristics of the soil at the base (see Section 12.13.3). Similarly, pinned column bases in frame-type structures are sometimes used to conservatively account for soil flexibility under an assumed rigid foundation element. Period shifting of a fixed-base model of a structure caused by soil-structure interaction is permitted in accordance with Chapter 19.

The fundamental mode of a structure with a geometrically complex arrangement of seismic force-resisting systems determined with a three-dimensional model may be associated to the torsional mode of response of the system, with mass participating in both horizontal directions (orthogonal) concurrently. The analyst must be attentive to this mass participation and recognize that the period used to compute the design base shear should be associated to the mode with the largest mass participation in the direction being considered. Often in this situation, these periods are close to each other. Significant separation between the torsional mode period (when fundamental) and the shortest translational mode period may be an indicator of an ill-conceived structural system or potential modeling error. The standard requires that the fundamental period, T, used to determine the design base shear, V, does not exceed the approximate fundamental period, T_a, times the upper limit coefficient, C_u, provided in Table 12.8-1. This period limit prevents the use of an unusually low base shear for design of a structure that is, analytically, overly flexible because of mass and stiffness inaccuracies in the analytical model. C_u has two effects on T_a. First, recognizing that project-specific design requirements and design assumptions can influence T, C_u lessens the conservatism inherent in the empirical formulas for T_a to more closely follow the mean curve (Fig. C12.8-2). Second, the values for C_u recognize that the

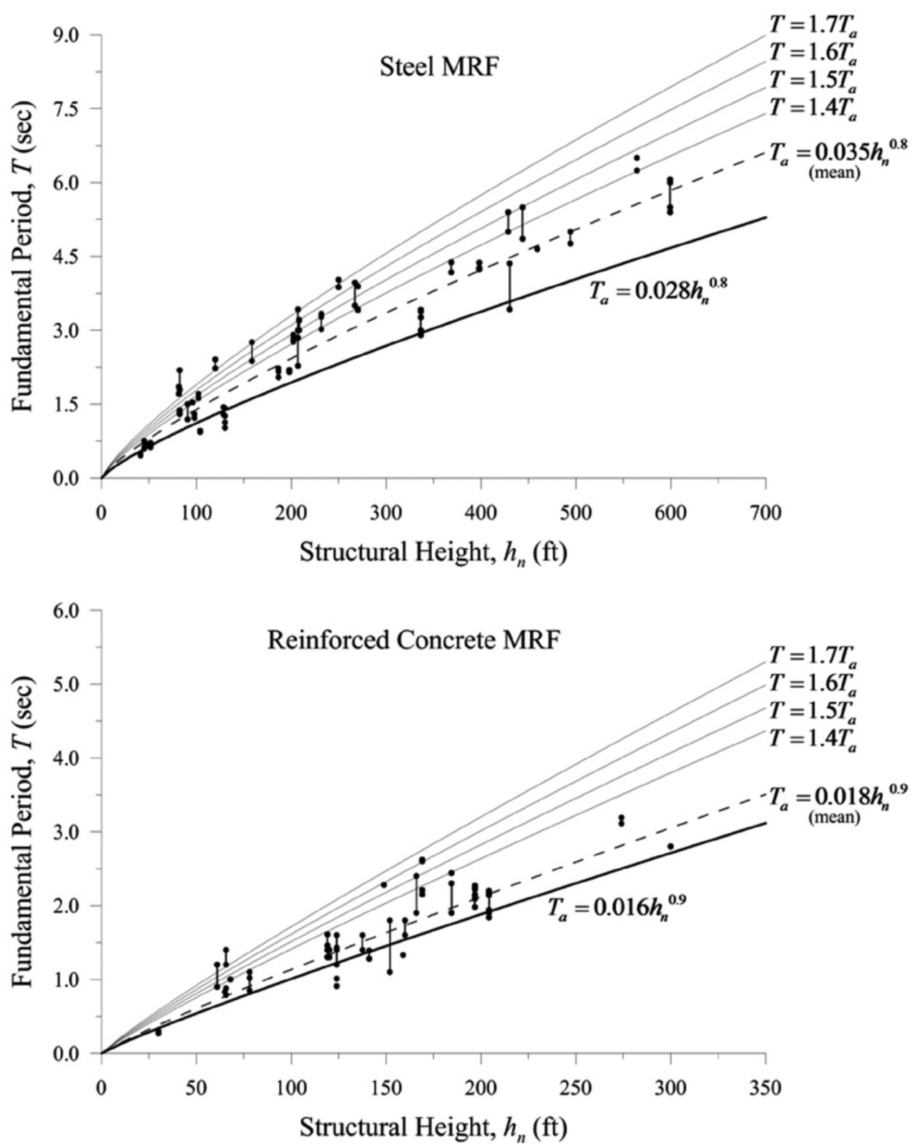

FIGURE C12.8-2 Variation of Fundamental Period with Structural Height

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formulas for T_a are targeted to structures in high seismic hazard locations. The stiffness of a structure is most likely to decrease in areas of lower seismicity, and this decrease is accounted for in the values of C_u. The response modification coefficient, R, typically decreases to account for reduced ductility demands, and the relative wind effects increase in lower seismic hazard locations. The design engineer must therefore be attentive to the value used for design of seismic force-resisting systems in structures that are controlled by wind effects. Although the value for C_u is most likely to be independent of the governing design forces in high wind areas, project-specific serviceability requirements may add considerable stiffness to a structure and decrease the value of C_u from considering seismic effects alone. This effect should be assessed where design forces for seismic and wind effects are almost equal. Lastly, if T from a properly substantiated analysis (Section 12.8.2) is less than C_uT_a, then the lower value of T and C_uT_a should be used for the design of the structure.

C12.8.2.1 Approximate Fundamental Period. Equation 12.8-7 is an empirical relationship determined through statistical analysis of the measured response of building structures in small- to moderate-sized earthquakes, including response to wind effects (Goel and Chopra 1997 and 1998). Figure C12.8-2 illustrates such data for various building structures with steel and reinforced concrete moment-resisting frames. Historically, the exponent, x, in Eq. 12.8-7 has been taken as 0.75 and was based on the assumption of a linearly varying mode shape while using Rayleigh's method. The exponents provided in the standard, however, are based on actual response data from building structures, thus more accurately reflecting the influence of mode shape on the exponent. Because the empirical expression is based on the lower bound of the data, it produces a lower bound estimate of the period for a building structure of a given height. This lower bound period, when used in Eqs. 12.8-3 and 12.8-4 to compute the seismic response coefficient, C_s, provides a conservative estimate of the seismic base shear, V.

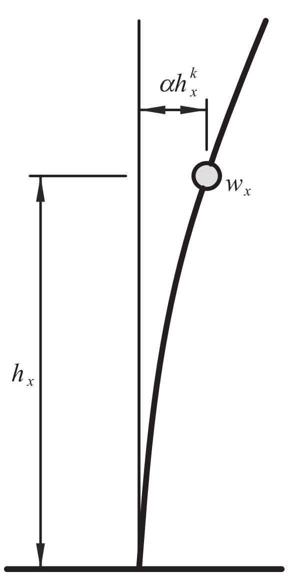
C12.8.3 Vertical Distribution of Seismic Forces. Equation 12.8-12 is based on the simplified first mode shape shown in Fig. C12.8-3. In the figure, F_x is the inertial force at level x, which is simply the absolute acceleration at level x times the mass at level x. The base shear is the sum of these inertial forces, and Eq. 12.8-11 simply gives the ratio of the lateral seismic force at level x, F_x, to the total design lateral force or shear at the base, V.

The deformed shape of the structure in Fig. C12.8-3 is a function of the exponent k, which is related to the fundamental period of the structure, T. The variation of k with T is illustrated in Fig. C12.8-4. The exponent k is intended to approximate the effect of higher modes, which are generally more dominant in structures with a longer fundamental period of vibration. Lopez and Cruz (1996) discuss the factors that influence higher modes of response. Although the actual first mode shape for a structure is also a function of the type of seismic force-resisting system, that effect is not reflected in these equations. Also, because T is limited to C_uT_a for design, this mode shape may differ from that corresponding to the statistically based empirical formula for the approximate fundamental period, T_a. A drift analysis in accordance with Section 12.8.6 can be conducted using the actual period (see C12.8.6). As such, k changes to account for the variation between T and the actual period.

The horizontal forces computed using Eq. 12.8-11 do not reflect the actual inertial forces imparted on a structure at any particular point in time. Instead, they are intended to provide lateral seismic forces at individual levels that are consistent with enveloped results from more accurate analyses (Chopra and Newmark 1980).

C12.8.4 Horizontal Distribution of Forces. Within the context of an ELF analysis, the horizontal distribution of lateral forces in a given story to various seismic force-resisting elements in that story depends on the type, geometric arrangement, and vertical extents of the structural elements and on the shape and flexibility of the floor or roof diaphragm. Because some elements of the seismic force-resisting system are expected to respond inelastically to the design ground motion, the distribution of forces to the various structural elements and other systems also depends on the strength of the yielding elements and their sequence of yielding (see Section C12.1.1). Such effects cannot be captured accurately by a linear elastic static analysis (Paulay 1997), and a nonlinear dynamic analysis is too computationally cumbersome to be applied to the design of most buildings. As such, approximate methods are used to account for uncertainties in horizontal distribution in an elastic static analysis, and to a lesser extent in elastic dynamic analysis.

Of particular concern in regard to the horizontal distribution of lateral forces is the torsional response of the structure during the earthquake. The standard requires that the inherent torsional moment be evaluated for every structure with diaphragms that are not flexible (see Section C12.8.4.1). Although primarily a factor for torsionally irregular structures, this mode of response has also been observed in structures that are designed to be



$$F_x = \omega^2 \alpha h_x^k \frac{w_x}{g}$$

$$V_b = \sum_{i=1}^{n} \omega^2 \alpha h_i^k \frac{w_i}{g}$$

$$C_{vx} = \frac{F_x}{V_b} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

FIGURE C12.8-3 Basis of Eq. 12.8-12

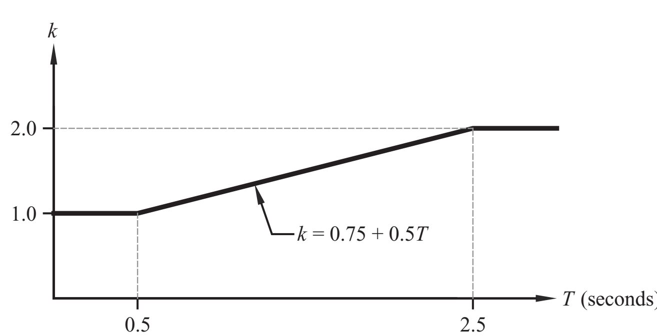


FIGURE C12.8-4 Variation of Exponent *k* with Period *T*

symmetric in plan and layout of seismic force-resisting systems (De La Llera and Chopra 1994). This torsional response in the case of a torsionally regular structure is caused by a variety of "accidental" torsional moments caused by increased eccentricities between the centers of rigidity and mass that exist because of uncertainties in quantifying the mass and stiffness distribution of the structure, as well as torsional components of earthquake ground motion that are not included explicitly in code-based designs (Newmark and Rosenbleuth 1971). Consequently, the accidental torsional moment can affect any structure, and potentially more so for a torsionally irregular structure. The standard requires that the accidental torsional moment be considered for every structure (see Section C12.8.4.2) as well as the amplification of this torsion for structures with torsional irregularity (see Section C12.8.4.3).

C12.8.4.1 Inherent Torsion. Where a rigid diaphragm is in the analytical model, the mass tributary to that floor or roof can be idealized as a lumped mass located at the resultant location on the floor or roof—termed the center of mass (CoM). This point represents the resultant of the inertial forces on the floor or roof. This diaphragm model simplifies structural analysis by reducing what would be many degrees of freedom in the two principal directions of a structure to three degrees of freedom (two horizontal and one rotational about the vertical axis). Similarly, the resultant stiffness of the structural members providing lateral stiffness to the structure tributary to a given floor or roof can be idealized as the center of rigidity (CoR).

It is difficult to accurately determine the center of rigidity for a multistory building because the center of rigidity for a particular story depends on the configuration of the seismic force-resisting elements above and below that story and may be load dependent (Chopra and Goel 1991). Furthermore, the location of the CoR is more sensitive to inelastic behavior than the CoM. If the CoM of a given floor or roof does not coincide with the CoR of that floor or roof, an inherent torsional moment, M_t, is created by the eccentricity between the resultant seismic force and the CoR. In addition to this *idealized* inherent torsional moment, the standard requires an accidental torsional moment, M_{ta}, be considered (see Section C12.8.4.2).

Similar principles can be applied to models of semirigid diaphragms that explicitly model the in-plane stiffness of the diaphragm, except that the deformation of the diaphragm needs to be included in computing the distribution of the resultant seismic force and inherent torsional moment to the seismic force-resisting system.

This inherent torsion is included automatically when performing a three-dimensional analysis using either a rigid or semirigid diaphragm. If a two-dimensional planar analysis is used, where permitted, the CoR and CoM for each story must be determined explicitly and the applied seismic forces must be adjusted accordingly.

For structures with flexible diaphragms (as defined in Section 12.3), vertical elements of the seismic force-resisting system are assumed to resist inertial forces from the mass that is tributary to the elements with no explicitly computed torsion. No diaphragm is perfectly flexible, therefore some torsional forces develop even when they are neglected.

C12.8.4.2 Accidental Torsion. The locations of the centers of mass and rigidity for a given floor or roof typically cannot be established with a high degree of accuracy because of mass and stiffness uncertainty and deviations in design, construction, and loading from the idealized case. To account for this inaccuracy, the standard requires the consideration of a minimum eccentricity of 5% of the width of a structure perpendicular to the direction being considered to any static eccentricity computed using idealized locations of the centers of mass and rigidity. Where a structure has a geometrically complex or nonrectangular floor plan, the eccentricity is computed using the diaphragm extents perpendicular to the direction of loading (see Section C12.5).

One approach to account for this variation in eccentricity is to shift the CoM each way from its calculated location and apply the seismic lateral force at each shifted location as separate seismic load cases. It is typically conservative to assume that the CoM offsets at all floors and roof occur simultaneously and in the same direction. This offset produces an "accidental" static torsional moment, M_{ta}, at each story. Most computer programs can automate this offset for three-dimensional analysis by automatically applying these static moments in the autogenerated seismic load case (along the global coordinate axes used in the computer model—see Section C12.5). Alternatively, user-defined torsional moments can be applied as separate load cases and then added to the seismic lateral force load case. For two-dimensional analysis, the accidental torsional moment is distributed to each seismic force-resisting system as an applied static lateral force in proportion to its relative elastic lateral stiffness and distance from the CoR.

Shifting the CoM is a static approximation and thus does not affect the dynamic characteristics of the structure, as would be the case were the CoM to be physically moved by, for example, altering the horizontal mass distribution and mass moment of inertia. Although this "dynamic" approach can be used to adjust the eccentricity, it can be too computationally cumbersome for static analysis and therefore is reserved for dynamic analysis (see Section C12.9.5).

The previous discussion is applicable only to a rigid diaphragm model. A similar approach can be used for a semirigid diaphragm model except that the accidental torsional moment is decoupled into nodal moments or forces that are placed throughout the diaphragm. The amount of nodal actions depends on how sensitive the diaphragm is to in-plane deformation. As the in-plane stiffness of the diaphragm decreases, tending toward a flexible diaphragm, the nodal inputs decrease proportionally.

The physical significance of this mass eccentricity should not be confused with the physical meaning of the eccentricity required for representing nonuniform wind pressures acting on a structure. However, this accidental torsion also incorporates to a lesser extent the potential torsional motion input into structures with large footprints from differences in ground motion within the footprint of the structure.

Torsionally irregular structures whose fundamental mode is potentially dominated by the torsional mode of response can be more sensitive to dynamic amplification of this accidental torsional moment. Consequently, the 5% minimum can underestimate the accidental torsional moment. In these cases, the standard requires the amplification of this moment for design when using an elastic static analysis procedure, including satisfying the drift limitations (see Section C12.8.4.3).

Accidental torsion results in forces that are combined with those obtained from the application of the seismic design story shears, V_x, including inherent torsional moments. All elements are designed for the maximum effects determined, considering positive accidental torsion, negative accidental torsion, and no accidental torsion (see Section C12.5). Where consideration of earthquake forces applied concurrently in any two orthogonal directions is required by the standard, it is permitted to apply the 5% eccentricity of the center of mass along the single orthogonal direction that produces the greater effect, but it need not be applied simultaneously in the orthogonal direction.

C12.8.4.3 Amplification of Accidental Torsional Moment. For structures with torsional or extreme torsional irregularity (Type 1a or 1b horizontal structural irregularity) analyzed using the equivalent lateral force procedure, the standard requires amplification of the accidental torsional moment to account for increases in the torsional moment caused by potential yielding of the perimeter seismic force-resisting systems (i.e., shifting of the center of rigidity), as well as other factors potentially leading to dynamic torsional instability. For verifying torsional irregularity requirements in Table 12.3-1, story drifts resulting from the applied loads, which include both the inherent and accidental torsional moments, are used with no amplification of the accidental torsional moment ($A_x = 1$). The same process is used when computing the amplification factor, A_x, except that displacements (relative to the base) at the level being evaluated are used in lieu of story drifts. Displacements are used here to indicate that amplification of the accidental torsional moment is primarily a system-level phenomenon, proportional to the increase in acceleration at the extreme edge of the structure, and not explicitly related to an individual story and the components of the seismic force-resisting system contained therein.

Equation 12.8-14 was developed by the SEAOC Seismology Committee to encourage engineers to design buildings with good torsional stiffness; it was first introduced in the UBC-88. Figure C12.8-5 illustrates the effect of Eq. 12.8-14 for a symmetric rectangular building with various aspect ratios (L/B) where the seismic force-resisting elements are positioned at a variable distance (defined by α) from the center of mass in each direction. Each element is assumed to have the same stiffness. The structure is loaded parallel to the short direction with an eccentricity of $0.05L$.

For α equal to 0.5, these elements are at the perimeter of the building, and for α equal to 0.0, they are at the center (providing no torsional resistance). For a square building ($L/B = 1.00$), A_x is greater than 1.0 where α is less than 0.25 and increases to its maximum value of 3.0 where α is equal to 0.11. For a rectangular building with L/B equal to 4.00, A_x is greater than 1.0 where α is less than 0.34 and increases to its maximum value of 3.0 where α is equal to 0.15.

C12.8.5 Overturning. The overturning effect on a vertical lateral force-resisting element is computed based on the calculation of lateral seismic force, F_x, times the height from the base to the level of the horizontal lateral force-resisting element that transfers F_x to the vertical element, summed over each story. Each vertical lateral force-resisting element resists its portion of overturning based on its relative stiffness with respect to all vertical lateral force-resisting elements in a building or structure. The seismic forces used are those from the equivalent lateral force procedure determined in Section 12.8.3 or based on a dynamic analysis of the building or structure. The overturning forces may be resisted by dead loads and can be combined with dead and live loads or other loads, in accordance with the load combinations of Section 12.4.2.3.

C12.8.6 Story Drift Determination. Equation 12.8-15 is used to estimate inelastic deflections (δ_x), which are then used to calculate design story drifts, Δ. These story drifts must be less than the allowable story drifts, Δ_a, of Table 12.12-1. For structures without torsional irregularity, computations are performed using deflections of the centers of mass of the floors bounding the story. If the eccentricity between the centers of mass of two adjacent floors, or a floor and a roof, is more than 5% of the width of the diaphragm extents, it is permitted to compute the deflection for the bottom of the story at the point on the floor that is vertically aligned with the location of the center of mass of the top floor or roof. This situation can arise where a building has story offsets and the diaphragm extents of the top of the story are smaller than the extents of the bottom of the story. For structures assigned to Seismic Design Category C, D, E, or F that are torsionally irregular, the standard requires that deflections be computed along the edges of the diaphragm extents using two vertically aligned points.

Figure C12.8-6 illustrates the force-displacement relationships between elastic response, response to reduced design-level forces, and the expected inelastic response. If the structure remained elastic during an earthquake, the force developed would be V_E, and the corresponding displacement would be δ_E. V_E does not include R, which accounts primarily for ductility and system overstrength. According to the equal displacement approximation rule of seismic response, the maximum displacement of an inelastic system is approximately equal to that of an elastic system with the same initial stiffness. This condition has been observed for structures idealized with bilinear inelastic

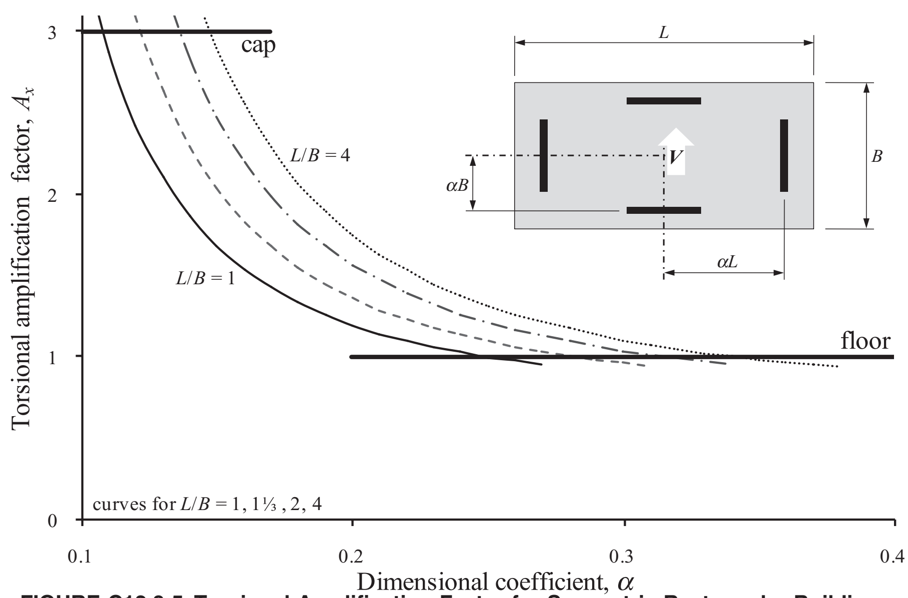


FIGURE C12.8-5 Torsional Amplification Factor for Symmetric Rectangular Buildings

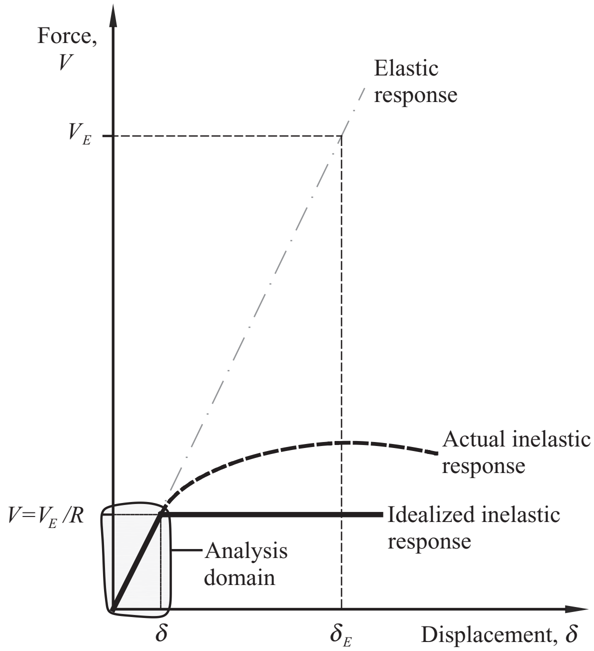


FIGURE C12.8-6 Displacements Used to Compute Drift

response and a fundamental period, T, greater than T_s (see Section 11.4.5). For shorter period structures, peak displacement of an inelastic system tends to exceed that of the corresponding elastic system. Because the forces are reduced by R, the resulting displacements are representative of an elastic system and need to be amplified to account for inelastic response.

The deflection amplification factor, C_d, in Eq. 12.8-15 amplifies the displacements computed from an elastic analysis using prescribed forces to represent the expected inelastic displacement for the design-level earthquake, and is typically less than R (Section C12.1.1). It is important to note that C_d is a story-level amplification factor and does not represent displacement amplification of the elastic response of a structure, either modeled as an *effective* single-degree-of-freedom structure (fundamental mode) or a constant amplification to represent the deflected shape of a multiple-degree-of-freedom structure, in effect, implying that the mode shapes do not change during inelastic response. Furthermore, drift-level forces are different than design-level forces used for strength compliance of the structural elements. Drift forces are typically lower as the computed fundamental period can be used to compute the base shear (see Section C12.8.6.2).

When conducting a drift analysis, the analyst should be attentive to the applied gravity loads used in combination with the strength-level earthquake forces so that consistency between the forces used in the drift analysis and those used for stability verification (P-Δ) in Section 12.8.7 is maintained, including consistency in computing the fundamental period if a second-order analysis is used. Further discussion is provided in C12.8.7.

The design forces used to compute the elastic deflection (δ_{xe}) include the importance factor, I_e, so Eq. 12.8-15 includes I_e in the denominator. This inclusion is appropriate because the allowable story drifts (except for masonry shear wall structures) in Table 12.12-1 are more stringent for higher risk categories.

C12.8.6.1 Minimum Base Shear for Computing Drift. Except for period limits (as described in Section C12.8.6.2), all of the requirements of Section 12.8 must be satisfied when computing drift for an ELF analysis, except that the minimum base shear

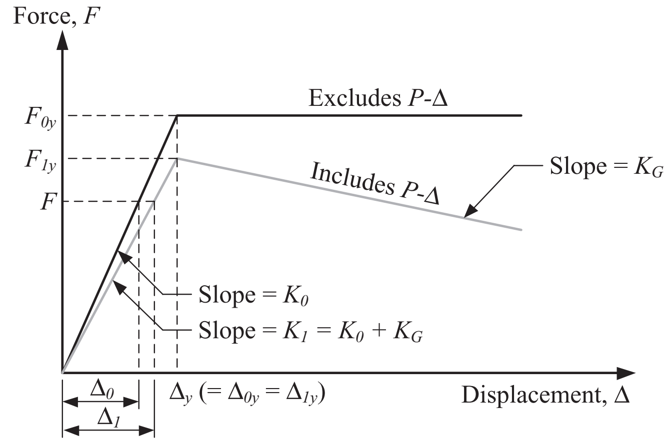


FIGURE C12.8-7 Idealized Response of a One-Story Structure with and without *P*-Δ

determined from applying Eq. 12.8-5 does not need to be considered. This equation represents a minimum strength that needs to be provided to a system (see Section C12.8.1.1). Equation 12.8-6 needs to be considered, when triggered, because it represents the increase in the response spectrum in the long-period range from near-fault effects.

C12.8.6.2 Period for Computing Drift. Where the design response spectrum of Section 11.4.5 or the corresponding equations of Section 12.8.1 are used and the fundamental period of the structure, T, is less than the long-period transition period, T_L, displacements increase with increasing period (even though forces may decrease). Section 12.8.2 applies an upper limit on T so that design forces are not underestimated, but if the lateral forces used to compute drifts are inconsistent with the forces corresponding to T, then displacements can be overestimated. To account for this variation in dynamic response, the standard allows the determination of displacements using forces that are consistent with the computed fundamental period of the structure without the upper limit of Section 12.8.2.

The analyst must still be attentive to the period used to compute drift forces. The same analytical representation (see Section C12.7.3) of the structure used for strength design must also be used for computing displacements. Similarly, the same analysis method (Table 12.6-1) used to compute design forces must also be used to compute drift forces. It is generally appropriate to use 85% of the computed fundamental period to account for mass and stiffness inaccuracies as a precaution against overly flexible structures, but need not be taken as less than that used for strength design. The more flexible the structure, the more likely it is that P-delta effects ultimately control the design (see Section C12.8.7). Computed values of T that are significantly greater than (perhaps more than 1.5 times in high seismic areas) C_uT_a may indicate a modeling error. Similar to the discussion in Section C12.8.2, the analyst should assess the value of C_u used where serviceability constraints from wind effects add significant stiffness to the structure.

C12.8.7 P-Delta Effects. Figure C12.8-7 shows an idealized static force-displacement response for a simple one-story structure (e.g., idealized as an inverted pendulum-type structure). As the top of the structure displaces laterally, the gravity load, P, supported by the structure acts through that displacement and produces an increase in overturning moment by P times the story drift, Δ, that must be resisted by the structure—the so-called

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"P-delta (P-Δ) effect." This effect also influences the lateral displacement response of the structure from an applied lateral force, F.

The response of the structure not considering the P-Δ effect is depicted by Condition 0 in the figure with a slope of K_0 and lateral first-order yield force F_{0y}. This condition characterizes the first-order response of the structure (the response of the structure from an analysis not including P-delta effects). Where the P-Δ effect is included (depicted by Condition 1 in the figure), the related quantities are K_1 and F_{1y}. This condition characterizes the second-order response of the structure (the response of the structure from an analysis including P-delta effects).

The geometric stiffness of the structure, K_G, in this example is equal to the gravity load, P, divided by the story height, h_{sx}. K_G is used to represent the change in lateral response by analytically reducing the elastic stiffness, K_0. K_G is negative where gravity loads cause compression in the structure. Because the two response conditions in the figure are for the same structure, the inherent yield displacement of the structure is the same ($\Delta_{0y} = \Delta_{1y} = \Delta_y$).

Two consequential points taken from the figure are (1) the increase in required strength and stiffness of the seismic force-resisting system where the P-Δ effect influences the lateral response of the structure must be accounted for in design, and (2) the P-Δ effect can create a negative stiffness condition during postyield response, which could initiate instability of the structure. Where the postyield stiffness of the structure may become negative, dynamic displacement demands can increase significantly (Gupta and Krawinkler 2000).

One approach that can be used to assess the influence of the P-Δ effect on the lateral response of a structure is to compare the first-order response to the second-order response, which can be done using an elastic stability coefficient, θ, defined as the absolute value of K_G divided by K_0.

$$\theta = \frac{|K_G|}{K_0} = \left|\frac{P\Delta_{0y}}{F_{0y}h_{sx}}\right| \qquad \text{(C12.8-1)}$$

Given the above, and the geometric relationships shown in Fig. C12.8-7, it can be shown that the force producing yield in condition 1 (with P-Δ effects) is

$$F_{1y} = F_{0y}(1 - \theta) \qquad \text{(C12.8-2)}$$

and that for a force, F, less than or equal to F_{1y}

$$\Delta_1 = \frac{\Delta_0}{1-\theta} \qquad \text{(C12.8-3)}$$

Therefore, the stiffness ratio, K_0/K_1, is

$$\frac{K_0}{K_1} = \frac{1}{1-\theta} \qquad \text{(C12.8-4)}$$

In the previous equations,

- F_{0y} = the lateral first-order yield force;
- F_{1y} = the lateral second-order yield force;
- h_{sx} = the story height (or structure height in this example);
- K_G = the geometric stiffness;
- K_0 = the elastic first-order stiffness;
- K_1 = the elastic second-order stiffness;
- P = the total gravity load supported by the structure;
- Δ_0 = the lateral first-order drift;
- Δ_{0y} = the lateral first-order yield drift;
- Δ_1 = the lateral second-order drift;
- Δ_{1y} = the lateral second-order yield drift; and
- θ = the elastic stability coefficient.

A physical interpretation of this effect is that to achieve the second-order response depicted in the figure, the seismic force-resisting system must be designed to have the increased stiffness and strength depicted by the first-order response. As θ approaches unity, Δ_1 approaches infinity and F_1 approaches zero, defining a state of static instability.

The intent of Section 12.8.7 is to determine whether P-Δ effects are significant when considering the first-order response of a structure and, if so, to increase the strength and stiffness of the structure to account for P-Δ effects. Some material-specific design standards require P-Δ effects to always be included in the elastic analysis of a structure and strength design of its members. The amplification of first-order member forces in accordance with Section 12.8.7 should not be misinterpreted to mean that these other requirements can be disregarded, nor should they be applied concurrently. Therefore, Section 12.8.7 is primarily used to verify compliance with the allowable drifts and check against potential postearthquake instability of the structure, while provisions in material-specific design standards are used to increase member forces for design, if provided. In so doing, the analyst should be attentive to the stiffness of each member used in the mathematical model so that synergy between standards is maintained.

Equation 12.8-16 is used to determine the elastic stability coefficient, θ, of each story of a structure.

$$\theta = \left|\frac{P\Delta_0}{F_0 h_{sx}}\right| = \frac{P\Delta I_e}{V_x h_{sx} C_d} \qquad \text{(C12.8-5)}$$

where h_{sx}, I_e, and V_x are the same as defined in the standard and

- F_0 = the force in a story causing $\Delta_0 = \sum F_x = V_x$;
- Δ_0 = the elastic lateral story drift = $\Delta I_e/C_d$;
- Δ = the inelastic story drift determined in accordance with Section 12.8.6; and
- P = the total *point-in-time* gravity load supported by the structure.

Structures with θ less than 0.10 generally are expected to have a positive monotonic postyield stiffness. Where θ for any story exceeds 0.10, P-Δ effects must be considered for the entire structure using one of the two approaches in the standard. Either first-order displacements and member forces are multiplied by $1/(1 - \theta)$ or the P-Δ effect is explicitly included in the structural analysis and the resulting θ is multiplied by $1/(1+\theta)$ to verify compliance with the first-order stability limit. Most commercial computer programs can perform second-order analysis. The analyst must therefore be attentive to the algorithm incorporated in the software and cognizant of any limitations, including suitability of iterative and noniterative methods, inclusion of second-order effects (P-Δ and P-δ) in automated modal analyses, and appropriateness of superposition of design forces.

Gravity load drives the increase in lateral displacements from the equivalent lateral forces. The standard requires the total vertical design load, and the largest vertical design load for combination with earthquake loads is given by combination 5 from Section 2.3.2, which is transformed in Section 12.4.2.3 to be

$$(1.2 + 0.2S_{DS})D + 1.0L + 0.2S + 1.0E$$

where the 1.0 factor on L is actually 0.5 for many common occupancies. The provision of Section 12.8.7 allows the factor

on dead load D to be reduced to 1.0 for the purpose of P-delta analysis under seismic loads. The vertical seismic component need not be considered for checking θ_{max}.

As explained in the commentary for Chapter 2, the 0.5 and 0.2 factors on L and S, respectively, are intended to capture the arbitrary point-in-time values of those loads. The factor 1.0 results in the dead load effect being fairly close to best estimates of the arbitrary point-in-time value for dead load. L is defined in Chapter 4 of the standard to include the reduction in live load based on floor area. Many commercially available computer programs do not include live load reduction in the basic structural analysis. In such programs, live reduction is applied only in the checking of design criteria; this difference results in a conservative calculation with regard to the requirement of the standard.

The seismic story shear, V_x (in accordance with Section 12.8.4), used to compute θ includes the importance factor, I_e. Furthermore, the design story drift, Δ (in accordance with Section 12.8.6), does not include this factor. Therefore, I_e has been added to Eq. 12.8-16 to correct an apparent omission in previous editions of the standard. Nevertheless, the standard has always required V_x and Δ used in this equation to be those occurring simultaneously.

Equation 12.8-17 establishes the maximum stability coefficient, θ_{max}, permitted. The intent of this requirement is to protect structures from the possibility of instability triggered by post-earthquake residual deformation. The danger of such failures is real and may not be eliminated by apparently available overstrength. This problem is particularly true of structures designed in regions of lower seismicity.

For the idealized system shown in Figure C12.8-7, assume the maximum displacement is $C_d\Delta_0$. Assuming the unloading stiffness, K_u, is equal to the elastic stiffness, K_0, the residual displacement is

$$\left(C_d - \frac{1}{\beta}\right)\Delta_0 \qquad \text{(C12.8-6)}$$

Additionally, assume that there is a factor of safety, FS, of 2 against instability at the maximum residual drift, $\Delta_{r,\max}$. Evaluating the overturning and resisting moments ($F_0 = V_0$ in this example),

$$P\Delta_{r,\max} \le \frac{V_0}{\beta FS}h \quad \text{where} \quad \beta = \frac{V_0}{V_{0y}} \le 1.0 \qquad \text{(C12.8-7)}$$

Therefore,

$$\frac{P[\Delta_0(\beta C_d - 1)]}{V_0 h} \le 0.5 \rightarrow \theta_{\max}(\beta C_d - 1) = 0.5 \rightarrow \theta_{\max} = \frac{0.5}{\beta C_d - 1}$$

Conservatively assume that $\beta C_d - 1 \approx \beta C_d$

$$\theta_{\max} = \frac{0.5}{\beta C_d} \le 0.25 \qquad \text{(C12.8-9)}$$

In the previous equations,

C_d = the displacement amplification factor;
FS = the factor of safety;
h_{sx} = the story height (or height of the structure in this example);
P = the total point-in-time gravity load supported by the structure;
V_0 = the first-order story shear demand;
V_{0y} = the first-order yield strength of the story;
β = the ratio of shear demand to shear capacity;
Δ_0 = the elastic lateral story drift;
$\Delta_{r,\max}$ = the maximum residual drift at $V_0 = 0$; and
$\theta_{\max}$ = the maximum elastic stability coefficient.

The standard requires that the computed stability coefficient, θ, not exceed 0.25 or $0.5/\beta C_d$, where βC_d is an adjusted ductility demand that takes into account the variation between the story strength demand and the story strength supplied. The story strength demand is simply V_x. The story strength supplied may be computed as the shear in the story that occurs simultaneously with the attainment of the development of first significant yield of the overall structure. To compute first significant yield, the structure should be loaded with a seismic force pattern similar to that used to compute story strength demand and iteratively increased until first yield. Alternatively, a simple and conservative procedure is to compute the ratio of demand to strength for each member of the seismic force-resisting system in a particular story and then use the largest such ratio as β.

The principal reason for inclusion of β is to allow for a more equitable analysis of those structures in which substantial extra strength is provided, whether as a result of added stiffness for drift control, code-required wind resistance, or simply a feature of other aspects of the design. Some structures inherently possess more strength than required, but instability is not typically a concern. For many flexible structures, the proportions of the structural members are controlled by drift requirements rather than strength requirements; consequently, β is less than 1.0 because the members provided are larger and stronger than required. This has the effect of reducing the inelastic component of total seismic drift, and thus, β is placed as a factor on C_d.

Accurate evaluation of β would require consideration of all pertinent load combinations to find the maximum ratio of demand to capacity caused by seismic load effects in each member. A conservative simplification is to divide the total demand with seismic load effects included by the total capacity; this simplification covers all load combinations in which dead and live load effects add to seismic load effects. If a member is controlled by a load combination where dead load counteracts seismic load effects, to be correctly computed, β must be based only on the seismic component, not the total. The gravity load, P, in the P-Δ computation would be less in such a circumstance and, therefore, θ would be less. The importance of the counteracting load combination does have to be considered, but it rarely controls instability.

Although the P-Δ procedure in the standard reflects a simple static idealization as shown in Fig. C12.8-7, the real issue is one of dynamic stability. To adequately evaluate second-order effects during an earthquake, a nonlinear response history analysis should be performed that reflects variability of ground motions and system properties, including initial stiffness, strain hardening stiffness, initial strength, hysteretic behavior, and magnitude of point-in-time gravity load, P. Unfortunately, the dynamic response of structures is highly sensitive to such parameters, causing considerable dispersion to appear in the results (Vamvatsikos 2002). This dispersion, which increases dramatically with stability coefficient θ, is caused primarily by the incrementally increasing residual deformations (ratcheting) that occur during the response. Residual deformations may be controlled by increasing either the initial strength or the secondary stiffness. Gupta and Krawinkler (2000) give additional information.

C12.9 MODAL RESPONSE SPECTRUM ANALYSIS

In the modal response spectrum analysis method, the structure is decomposed into a number of single-degree-of-freedom systems, each having its own mode shape and natural period of vibration. The number of modes available is equal to the number of mass degrees of freedom of the structure, so the number of modes can be reduced by eliminating mass degrees of freedom. For example, rigid diaphragm constraints may be used to reduce the number of mass degrees of freedom to one per story for planar models, and to three per story (two translations and rotation about the vertical axis) for three-dimensional structures. However, where the vertical elements of the seismic force-resisting system have significant differences in lateral stiffness, rigid diaphragm models should be used with caution because relatively small in-plane diaphragm deformations can have a significant effect on the distribution of forces.

For a given direction of loading, the displacement in each mode is determined from the corresponding spectral acceleration, modal participation, and mode shape. Because the sign (positive or negative) and the time of occurrence of the maximum acceleration are lost in creating a response spectrum, there is no way to recombine modal responses exactly. However, statistical combination of modal responses produces reasonably accurate estimates of displacements and component forces. The loss of signs for computed quantities leads to problems in interpreting force results where seismic effects are combined with gravity effects, produce forces that are not in equilibrium, and make it impossible to plot deflected shapes of the structure.

C12.9.1 Number of Modes. The key motivation to perform modal response spectrum analysis is to determine how the actual distribution of mass and stiffness of a structure affects the elastic displacements and member forces. Where at least 90% of the model mass participates in the response, the distribution of forces and displacements is sufficient for design. The scaling required by Section 12.9.4 controls the overall magnitude of design values so that incomplete mass participation does not produce nonconservative results.

The number of modes required to achieve 90% modal mass participation is usually a small fraction of the total number of modes. Lopez and Cruz (1996) contribute further discussion of the number of modes to use for modal response spectrum analysis.

C12.9.2 Modal Response Parameters. The design response spectrum (whether the general spectrum from Section 11.4.5 or a site-specific spectrum determined in accordance with Section 21.2) is representative of linear elastic structures. Division of the spectral ordinates by the response modification coefficient, R, accounts for inelastic behavior, and multiplication of spectral ordinates by the importance factor, I_e, provides the additional strength needed to improve the performance of important structures. The displacements that are computed using the response spectrum that has been modified by R and I_e (for strength) must be amplified by C_d and reduced by I_e to produce the expected inelastic displacements (see Section C12.8.6.)

C12.9.3 Combined Response Parameters. Most computer programs provide for either the SRSS or the CQC method (Wilson et al. 1981) of modal combination. The two methods are identical where applied to planar structures, or where zero damping is specified for the computation of the cross-modal coefficients in the CQC method. The modal damping specified in each mode for the CQC method should be equal to the damping level that was used in the development of the design response spectrum. For the spectrum in Section 11.4.5, the damping ratio is 0.05.

The SRSS or CQC method is applied to loading in one direction at a time. Where Section 12.5 requires explicit consideration of orthogonal loading effects, the results from one direction of loading may be added to 30% of the results from loading in an orthogonal direction. Wilson (2000) suggests that a more accurate approach is to use the SRSS method to combine 100% of the results from each of two orthogonal directions where the individual directional results have been combined by SRSS or CQC, as appropriate.

The CQC-4 method (as modified by ASCE 4) is also specified and is an alternative to the required use of the CQC method where there are closely spaced modes with significant cross-correlation of translational and torsional response. The CQC-4 method varies slightly from the CQC method through the use of a parameter that forces a correlation in modal responses where they are partially or completely in phase with the input motion. This difference primarily affects structures with short fundamental periods, T, that have significant components of response that are in phase with the ground motion. In these cases, using the CQC method can be nonconservative. A general overview of the various modal response combination methods can be found in U.S Nuclear Regulatory Commission (1999).

C12.9.4 Scaling Design Values of Combined Response. The modal base shear, V_t, may be less than the ELF base shear, V, because (a) the calculated fundamental period, T, may be longer than that used in computing V; (b) the response is not characterized by a single mode; or (c) the ELF base shear assumes 100% mass participation in the first mode, which is always an overestimate.

C12.9.4.1 Scaling of Forces. The scaling required by Section 12.9.4.1 provides, in effect, a minimum base shear for design. This minimum base shear is provided because the computed fundamental period may be the result of an overly flexible (incorrect) analytical model. The possible 15% reduction in design base shear may be considered as an incentive for using a modal response spectrum analysis in lieu of the equivalent lateral force procedure.

C12.9.4.2 Scaling of Drifts. Displacements from the modal response spectrum are not scaled because the use of an overly flexible model results in conservative estimates of displacement that need not be further scaled.

C12.9.5 Horizontal Shear Distribution. Torsion effects in accordance with Section 12.8.4 must be included in the modal response spectrum analysis (MRSA) as specified in Section 12.9 by requiring use of the procedures in Section 12.8 for the determination of the seismic base shear, V. There are two basic approaches for consideration of accidental torsion.

The first approach follows the static procedure discussed in C12.8.4.2, where the total seismic lateral forces obtained from the MRSA—using the computed locations of the centers of mass and rigidity—are statically applied at an artificial point offset from the center of mass to compute the accidental torsional moments. Most computer programs can automate this procedure for three-dimensional analysis. Alternatively, the torsional moments can be statically applied as separate load cases and added to the results obtained from the MRSA.

Because this approach is a static approximation, amplification of the accidental torsion in accordance with Section 12.8.4.3 is required. MRSA results in a single, positive response, thus inhibiting direct assessment of torsional response. One method to circumvent this problem is to determine the maximum and

average displacements for each mode participating in the direction being considered and then apply modal combination rules (primarily the CQC method) to obtain the total displacements used to check torsional irregularity and compute the amplification factor, A_x. The analyst should be attentive about how accidental torsion is included for individual modal responses.

The second approach, which applies primarily to three-dimensional analysis, is to modify the dynamic characteristics of the structure so that dynamic amplification of the accidental torsion is directly considered. This modification can be done, for example, by either reassigning the lumped mass for each floor and roof (rigid diaphragm) to alternate points offset from the initially calculated center of mass and modifying the mass moment of inertia, or physically relocating the initially calculated center of mass on each floor and roof by modifying the horizontal mass distribution (typically presumed to be uniformly distributed). This approach increases the computational demand significantly because all possible configurations would have to be analyzed, primarily two additional analyses for each principal axis of the structure. The advantage of this approach is that the dynamic effects of direct loading and accidental torsion are assessed automatically. Practical disadvantages are the increased bookkeeping required to track multiple analyses and the cumbersome calculations of the mass properties.

Where this "dynamic" approach is used, amplification of the accidental torsion in accordance with Section 12.8.4.3 is not required because repositioning the center of mass increases the coupling between the torsional and lateral modal responses, directly capturing the amplification of the accidental torsion.

Most computer programs that include accidental torsion in a MRSA do so statically (first approach discussed above) and do not physically shift the center of mass. The designer should be aware of the methodology used for consideration of accidental torsion in the selected computer program.

C12.9.6 P-Delta Effects. The requirements of Section 12.8.7, including the stability coefficient limit, θ_{max}, apply to modal response spectrum analysis.

C12.9.7 Soil-Structure Interaction Reduction. The standard permits including soil-structure interaction (SSI) effects in a modal response spectrum analysis in accordance with Chapter 19. The increased use of modal analysis for design stems from computer analysis programs automatically performing such an analysis. However, common commercial programs do not give analysts the ability to customize modal response parameters. This problem hinders the ability to include SSI effects in an automated modal analysis.

C12.10 DIAPHRAGMS, CHORDS, AND COLLECTORS

C12.10.1 Diaphragm Design. Diaphragms are generally treated as horizontal deep beams or trusses that distribute lateral forces to the vertical elements of the seismic force-resisting system. As deep beams, diaphragms must be designed to resist the resultant shear and bending stresses. Diaphragms are commonly compared to girders, with the roof or floor deck analogous to the girder web in resisting shear, and the boundary elements (chords) analogous to the flanges of the girder in resisting flexural tension and compression. As in girder design, the chord members (flanges) must be sufficiently connected to the body of the diaphragm (web) to prevent separation and to force the diaphragm to work as a single unit.

Diaphragms may be considered flexible, semirigid, or rigid. The flexibility or rigidity of the diaphragm determines how lateral forces are distributed to the vertical elements of the seismic force-resisting system (see Section C12.3.1). Once the distribution of lateral forces is determined, shear and moment diagrams are used to compute the diaphragm shear and chord forces. Where diaphragms are not flexible, inherent and accidental torsion must be considered in accordance with Section 12.8.4.

Diaphragm openings may require additional localized reinforcement (subchords and collectors) to resist the subdiaphragm chord forces above and below the opening and to collect shear forces where the diaphragm depth is reduced (Fig. C12.10-1). Collectors on each side of the opening drag shear into the subdiaphragms above and below the opening. The subchord and collector reinforcement must extend far enough into the adjacent diaphragm to develop the axial force through shear transfer. The required development length is determined by dividing the axial force in the subchord by the shear capacity (in force/unit length) of the main diaphragm.

Chord reinforcement at reentrant corners must extend far enough into the main diaphragm to develop the chord force through shear transfer (Fig. C12.10-2). Continuity of the chord members also must be considered where the depth of the diaphragm is not constant.

In wood and metal deck diaphragm design, framing members are often used as continuity elements, serving as subchords and collector elements at discontinuities. These continuity members also are often used to transfer wall out-of-plane forces to the main diaphragm, where the diaphragm itself does not have the capacity to resist the anchorage force directly. For additional discussion, see Sections C12.11.2.2.3 and C12.11.2.2.4.

C12.10.1.1 Diaphragm Design Forces. Diaphragms must be designed to resist inertial forces, as specified in Eq. 12.10-1 and to transfer design seismic forces caused by horizontal offsets or changes in stiffness of the vertical resisting elements. Inertial forces are those seismic forces that originate at the specified diaphragm level, whereas the transfer forces originate above the

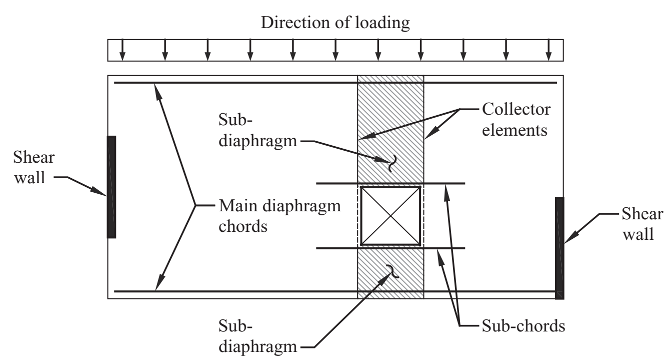

FIGURE C12.10-1 Diaphragm with an Opening

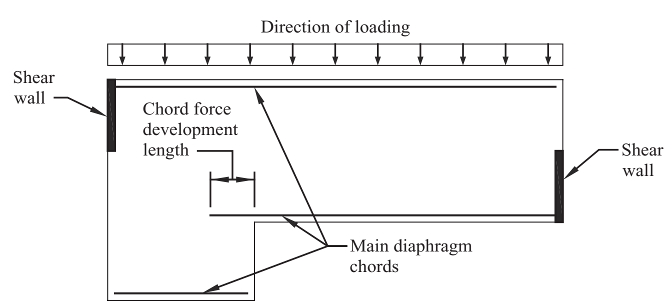

FIGURE C12.10-2 Diaphragm with a Reentrant Corner

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specified diaphragm level. The redundancy factor, ρ, used for design of the seismic force-resisting elements also applies to diaphragm transfer forces, thus completing the load path.

C12.10.2.1 Collector Elements Requiring Load Combinations with Overstrength Factor for Seismic Design Categories C through F. The overstrength requirement of this section is intended to keep inelastic behavior in the ductile elements of the seismic force-resisting system (consistent with the response modification coefficient, *R*) rather than in collector elements.

C12.11 STRUCTURAL WALLS AND THEIR ANCHORAGE

As discussed in Section C1.4, structural integrity is important not only in earthquake-resistant design but also in resisting high winds, floods, explosion, progressive failure, and even such ordinary hazards as foundation settlement. The detailed requirements of this section address wall-to-diaphragm integrity.

C12.11.1 Design for Out-of-Plane Forces. Because they are often subjected to local deformations caused by material shrinkage, temperature changes, and foundation movements, wall connections require some degree of ductility to accommodate slight movements while providing the required strength.

Although nonstructural walls are not subject to this requirement, they must be designed in accordance with Chapter 13.

C12.11.2 Anchorage of Structural Walls and Transfer of Design Forces into Diaphragms. There are numerous instances in U.S. earthquakes of tall, single-story, and heavy walls becoming detached from supporting roofs, resulting in collapse of walls and supported bays of roof framing (Hamburger and McCormick 2004). The response involves dynamic amplification of ground motion by response of vertical system and further dynamic amplification from flexible diaphragms. The design forces for Seismic Design Category D and higher have been developed over the years in response to studies of specific failures. It is generally accepted that the rigid diaphragm value is reasonable for structures subjected to high ground motions. For a simple idealization of the dynamic response, these values imply that the combined effects of inelastic action in the main framing system supporting the wall, the wall (acting out of plane), and the anchor itself correspond to a reduction factor of 4.5 from elastic response to an MCE motion, and therefore the value of the response modification coefficient, *R*, associated with nonlinear action in the wall or the anchor itself is 3.0. Such reduction is generally not achievable in the anchorage itself, thus it must come from yielding elsewhere in the structure, for example, the vertical elements of the seismic force-resisting system, the diaphragm, or walls acting out of plane. The minimum forces are based on the concept that less yielding occurs with smaller ground motions and less yielding is achievable for systems with smaller values of *R*, which are permitted in structures assigned to Seismic Design Categories B and C. The minimum value of *R* in structures assigned to Seismic Design Category D, except cantilever column systems and light-frame walls sheathed with materials other than wood structural panels, is 3.25, whereas the minimum values of *R* for Categories B and C are 1.5 and 2.0, respectively.

Where the roof framing is not perpendicular to anchored walls, provision needs to be made to transfer both the tension and sliding components of the anchorage force into the roof diaphragm. Where a wall cantilevers above its highest attachment to, or near, a higher level of the structure, the reduction factor based on the height within the structure, $(1 + 2z/h)/3$, may result in a lower anchorage force than appropriate. In such an instance, using a value of 1.0 for the reduction factor may be more appropriate.

C12.11.2.1 Wall Anchorage Forces. Diaphragm flexibility can amplify out-of-plane accelerations so that the wall anchorage forces in this condition are twice those defined in Section 12.11.1.

C12.11.2.2 Additional Requirements for Diaphragms in Structures Assigned to Seismic Design Categories C through F

C12.11.2.2.1 Transfer of Anchorage Forces into Diaphragm. This requirement, which aims to prevent the diaphragm from tearing apart during strong shaking by requiring transfer of anchorage forces across the complete depth of the diaphragm, was prompted by failures of connections between tilt-up concrete walls and wood panelized roof systems in the 1971 San Fernando earthquake.

Depending on diaphragm shape and member spacing, numerous suitable combinations of subdiaphragms and continuous tie elements and smaller sub-subdiaphragms connecting to larger subdiaphragms and continuous tie elements are possible. The configuration of each subdiaphragm (or sub-subdiaphragm) provided must comply with the simple 2.5-to-1 length-to-width ratio, and the continuous ties must have adequate member and connection strength to carry the accumulated wall anchorage forces.

C12.11.2.2.2 Steel Elements of Structural Wall Anchorage System. A multiplier of 1.4 has been specified for strength design of steel elements to obtain a fracture strength of almost 2 times the specified design force (where φ_t is 0.75 for tensile rupture).

C12.11.2.2.3 Wood Diaphragms. Material standards for wood structural panel diaphragms permit the sheathing to resist shear forces only; use of diaphragm sheathing to resist direct tension or compression forces is not permitted. Therefore, seismic out-of-plane anchorage forces from structural walls must be transferred into framing members (such as beams, purlins, or subpurlins) using suitable straps or anchors. For wood diaphragms, it is common to use local framing and sheathing elements as subdiaphragms to transfer the anchorage forces into more concentrated lines of drag or continuity framing that carry the forces across the diaphragm and hold the building together. Figure C12.11-1

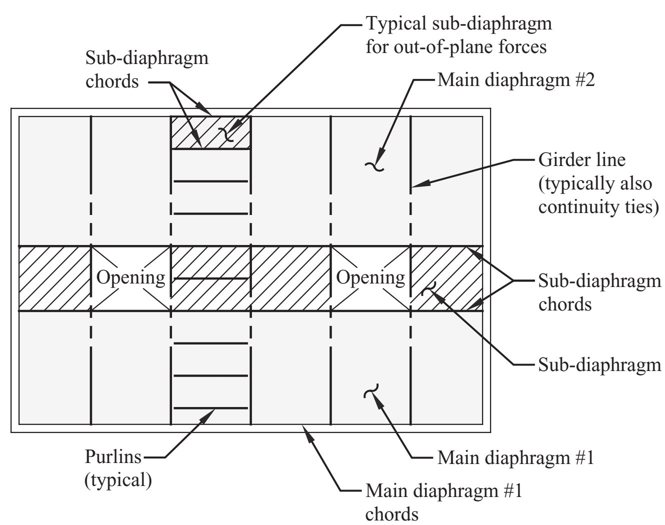

FIGURE C12.11-1 Typical Subdiaphragm Framing

shows a schematic plan of typical roof framing using subdiaphragms.

Fasteners that attach wood ledgers to structural walls are intended to resist shear forces from diaphragm sheathing attached to the ledger that act longitudinally along the length of the ledger but not shear forces that act transversely to the ledger, which tend to induce splitting in the ledger caused by cross-grain bending. Separate straps or anchors generally are provided to transfer out-of-plane wall forces into perpendicular framing members.

C12.11.2.2.4 Metal Deck Diaphragms. In addition to transferring shear forces, metal deck diaphragms often can resist direct axial forces in at least one direction. However, corrugated metal decks cannot transfer axial forces in the direction perpendicular to the corrugations and are prone to buckling if the unbraced length of the deck as a compression element is large. To manage diaphragm forces perpendicular to the deck corrugations, it is common for metal decks to be supported at 8- to 10-ft intervals by joists that are connected to walls in a manner suitable to resist the full wall anchorage design force and to carry that force across the diaphragm. In the direction parallel to the deck corrugations, subdiaphragm systems are considered near the walls; if the compression forces in the deck become large relative to the joist spacing, small compression reinforcing elements are provided to transfer the forces into the subdiaphragms.

C12.11.2.2.5 Embedded Straps. Steel straps may be used in systems where heavy structural walls are connected to wood or steel diaphragms as the wall-to-diaphragm connection system. In systems where steel straps are embedded in concrete or masonry walls, the straps are required to be bent around reinforcing bars in the walls, which improve their ductile performance in resisting earthquake load effects (e.g., the straps pull the bars out of the wall before the straps fail by pulling out without pulling the reinforcing bars out). Consideration should be given to the probability that light steel straps have been used in past earthquakes and have developed cracks or fractures at the wall-to-diaphragm framing interface because of gaps in the framing adjacent to the walls.

C12.11.2.2.6 Eccentrically Loaded Anchorage System. Wall anchors often are loaded eccentrically, either because the anchorage mechanism allows eccentricity, or because of anchor bolt or strap misalignment. This eccentricity reduces the anchorage connection capacity and hence must be considered explicitly in design of the anchorage. Figure C12.11-2 shows a one-sided roof-to-wall anchor that is subjected to severe eccentricity because of a misplaced anchor rod. If the detail were designed as a concentric two-sided connection, this condition would be easier to correct.

C12.11.2.2.7 Walls with Pilasters. The anchorage force at pilasters must be calculated considering two-way bending in wall panels. It is customary to anchor the walls to the diaphragms assuming one-way bending and simple supports at the top and bottom of the wall. However, where pilasters are present in the walls, their stiffening effect must be taken into account. The panels between pilasters are typically supported along all panel edges. Where this support occurs, the reaction at the top of the pilaster is the result of two-way action of the panel and is applied directly to the anchorage supporting the top of the pilaster. The anchor load at the pilaster generally is larger than the typical uniformly distributed anchor load between pilasters. Figure C12.11-3 shows the tributary area typically used to determine the anchorage force for a pilaster.

Anchor points adjacent to the pilaster must be designed for the full tributary loading, conservatively ignoring the effect of the adjacent pilaster.

C12.12 DRIFT AND DEFORMATION

As used in the standard, deflection is the absolute lateral displacement of any point in a structure relative to its base, and design story drift, Δ, is the difference in deflection along the height of a story (i.e., the deflection of a floor relative to that of the floor below). The drift, Δ, is calculated according to the procedures of Section 12.8.6. (Sections 12.9.2 and 16.1 give procedures for calculating displacements for modal response spectrum and linear response history analysis procedures, respectively; the definition of Δ in Section 11.3 should be used).

Calculated story drifts generally include torsional contributions to deflection (i.e., additional deflection at locations of the

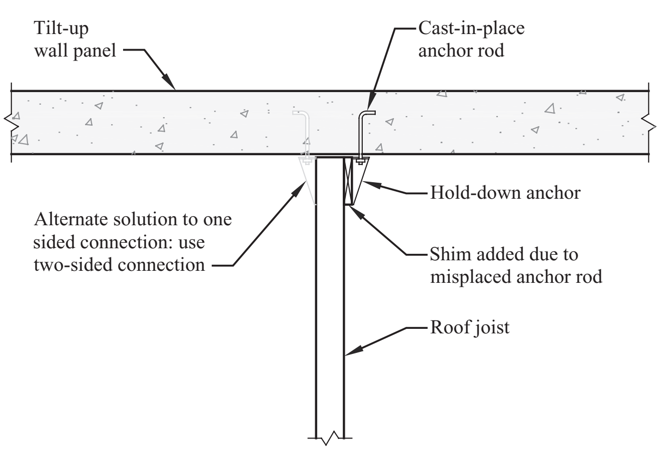

FIGURE C12.11-2 Plan View of Wall anchor with Misplaced Anchor Rod

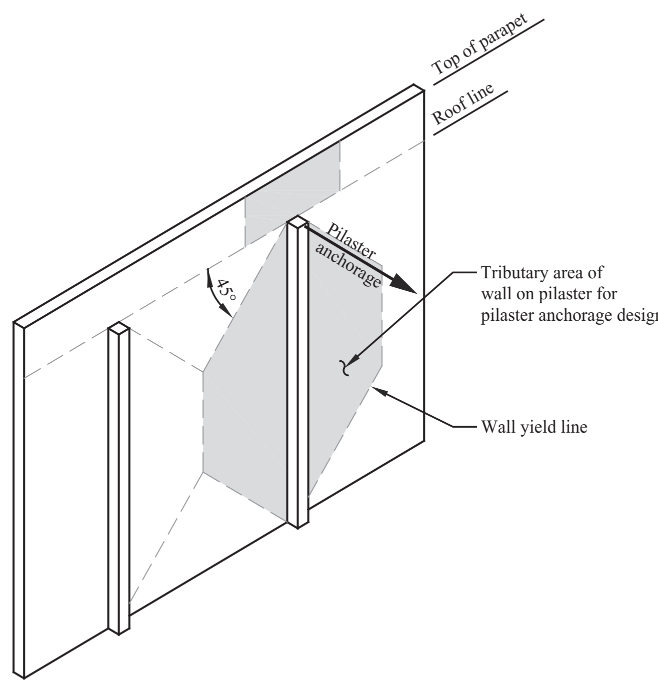

FIGURE C12.11-3 Tributary Area Used to Determine Anchorage Force at Pilaster

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center of rigidity at other than the center of mass caused by diaphragm rotation in the horizontal plane). The provisions allow these contributions to be neglected where they are not significant, such as in cases where the calculated drifts are much less than the allowable story drifts, Δ_a, no torsional irregularities exist, and more precise calculations are not required for structural separations (see Sections C12.12.3 and C12.12.4).

The deflections and design story drifts are calculated using for the design earthquake ground motion, which is two-thirds of the risk-targeted maximum considered earthquake (MCE_R) ground motion. The resulting drifts are therefore likely to be underestimated.

The design base shear, V, used to calculate Δ is reduced by the response modification coefficient, R. Multiplying displacements by the deflection amplification factor, C_d, is intended to correct for this reduction and approximate inelastic drifts corresponding to the design response spectrum unreduced by R. However, it is recognized that use of values of C_d less than R underestimates deflections (Uang and Maarouf 1994). Also Sections C12.8.6.2 and C12.9.4 deal with the appropriate base shear for computing displacements.

For these reasons, the displacements calculated may not correspond well to MCE_R ground motions. However, they are appropriate for use in evaluating the structure's compliance with the story drift limits put forth in Table 12.12-1 of the standard.

There are many reasons to limit drift; the most significant are to address the structural performance of member inelastic strain and system stability and to limit damage to nonstructural components, which can be life-threatening. Drifts provide a direct but imprecise measure of member strain and structural stability. Under small lateral deformations, secondary stresses caused by the P-delta effect are normally within tolerable limits (see Section C12.8.7). The drift limits provide indirect control of structural performance.

Buildings subjected to earthquakes need drift control to limit damage to partitions, shaft and stair enclosures, glass, and other fragile nonstructural components. The drift limits have been established without regard to economic considerations such as a comparison of present worth of future repairs with additional structural costs to limit drift. These are matters for building owners and designers to address.

The allowable story drifts, Δ_a, of Table 12.12-1 reflect the consensus opinion of the ASCE 7 Committee taking into account the life-safety and damage control objectives described in the aforementioned. Because the displacements induced in a structure include inelastic effects, structural damage as the result of a design-level earthquake is likely. This notion may be seen from the values of Δ_a stated in Table 12.12-1. For other structures assigned to Risk Category I or II, the value of Δ_a is 0.02 h_{sx}, which is about 10 times the drift ordinarily allowed under wind loads. If deformations well in excess of Δ_a were to occur repeatedly, structural elements of the seismic force-resisting system could lose so much stiffness or strength that they would compromise the safety and stability of the structure.

To provide better performance for structures assigned to Risk Category III or IV, their allowable story drifts, Δ_a, generally are more stringent than for those assigned to Risk Category I or II. However, those limits are still greater than the damage thresholds for most nonstructural components. Therefore, though the performance of structures assigned to Risk Category III or IV should be improved, there may be considerable damage from a design-level earthquake.

The allowable story drifts, Δ_a, for structures a maximum of four stories above the base are relaxed somewhat, provided the interior walls, partitions, ceilings, and exterior wall systems have been designed to accommodate story drifts. The type of structure envisioned by footnote c in Table 12.12-1 would be similar to a prefabricated steel structure with metal skin.

The values of Δ_a set forth in Table 12.12-1 apply to each story. For some structures, satisfying strength requirements may produce a system with adequate drift control. However, the design of moment-resisting frames and of tall, narrow shear walls or braced frames often is governed by drift considerations. Where design spectral response accelerations are large, seismic drift considerations are expected to control the design of midrise buildings.

C12.12.3 Structural Separation. This section addresses the potential for impact from adjacent structures during an earthquake. Such conditions may arise because of construction on or near a property line or because of the introduction of separations within a structure (typically called "seismic joints") for the purpose of permitting their independent response to earthquake ground motion. Such joints may effectively eliminate irregularities and large force transfers between portions of the building with different dynamic properties.

The standard requires the distance to be "sufficient to avoid damaging contact under total deflection." It is recommended that the distance be computed using the square root of the sum of the squares of the lateral deflections. Such a combination method treats the deformations as linearly independent variables. The deflections used are the expected displacements (e.g., the anticipated maximum inelastic deflections including the effects of torsion and diaphragm deformation). Just as these displacements increase with height, so does the required separation. If the effects of impact can be shown not to be detrimental, the required separation distances can be reduced.

For rigid shear wall structures with rigid diaphragms whose lateral deflections cannot be reasonably estimated, the NEHRP provisions (FEMA 2009a) suggest that older code requirements for structural separations of at least 1 in. (25 mm) plus 1/2 in. (13 mm) for each 10 ft (3 m) of height above 20 ft (6 m) be followed.

C12.12.4 Members Spanning between Structures. Where a portion of the structure is seismically separated from its support, the design of the support requires attention to ensure that support is maintained as the two portions move independently during earthquake ground motions. To prevent loss of gravity support for members that bridge between the two portions, the relative displacement must not be underestimated. Displacements computed for verifying compliance with drift limits (Eq. 12.8-15) and structural separations (Eq. 12.12-1) may be insufficient for this purpose.

The provision gives four requirements to ensure that displacement is not underestimated:

1. The deflections calculated using Eq. 12.8-15 are multiplied by $1.5R/C_d$ to correct for likely underestimation of displacement by the equation. The factor of 1.5 corrects for the 2/3 factor that is used in the calculation of seismic base shear, V, by reducing the base shear from the value based on the MCE_R ground motion (Section 11.4.4). Multiplying by R/C_d corrects for the fact that values of C_d less than R underestimate deflections (Uang and Maarouf 1994).
2. The deflections are calculated for torsional effects, including amplification factors. Diaphragm rotation can add significantly to the center-of-mass displacements calculated using Eq. 12.8-15.
3. Displacement caused by diaphragm deformations are required to be calculated, as in some types of construction

where the deformation during earthquake ground motions of the diaphragm can be considerable.

4. The absolute sum of displacements of the two portions is used instead of a modal combination, such as with Eq. 12.12-2, which would represent a probable value.

It is recognized that displacements so calculated are likely to be conservative. However, the consequences of loss of gravity support are likely to be severe and some conservatism is deemed appropriate.

C12.12.5 Deformation Compatibility for Seismic Design Categories D through F. In regions of high seismicity, many designers apply ductile detailing requirements to elements that are intended to resist seismic forces but neglect such practices for nonstructural components, or for structural components that are designed to resist only gravity forces but must undergo the same lateral deformations as the designated seismic force-resisting system. Even where elements of the structure are not intended to resist seismic forces and are not detailed for such resistance, they can participate in the response and may suffer severe damage as a result. This provision requires the designer to provide a level of ductile detailing or proportioning to all elements of the structure appropriate to the calculated deformation demands at the design story drift, Δ. This provision may be accomplished by applying details in gravity members similar to that used in members of the seismic force-resisting system or by providing sufficient *strength in those members, or by providing sufficient stiffness in the overall structure to preclude ductility demands in those members*.

In the 1994 Northridge earthquake, such participation was a cause of several failures. A preliminary reconnaissance report of that earthquake (EERI 1994) states:

Of much significance is the observation that six of the seven partial collapses (in modern precast concrete parking structures) seem to have been precipitated by damage to the gravity load system. Possibly, the combination of large lateral deformation and vertical load caused crushing in poorly confined columns that were not detailed to be part of the lateral load resisting system. Punching shear failures were observed in some structures at slab-to-column connections, such as at the Four Seasons building in Sherman Oaks. The primary lateral load resisting system was a perimeter ductile frame that performed quite well. However, the interior slab-column system was incapable of undergoing the same lateral deflections and experienced punching failures.

This section addresses such concerns. Rather than relying on designers to assume appropriate levels of stiffness, this section explicitly requires that the stiffening effects of adjoining rigid structural and nonstructural elements be considered and that a rational value of member and restraint stiffness be used for the design of structural components that are not part of the seismic force-resisting system.

This section also includes a requirement to address shears that can be induced in structural components that are not part of the seismic force-resisting system, because sudden shear failures have been catastrophic in past earthquakes.

The exception is intended to encourage the use of intermediate or special detailing in beams and columns that are not part of the seismic force-resisting system. In return for better detailing, such beams and columns are permitted to be designed to resist moments and shears from unamplified deflections. This design approach reflects observations and experimental evidence that well-detailed structural components can accommodate large drifts by responding inelastically without losing significant vertical load-carrying capacity.

C12.13 FOUNDATION DESIGN

C12.13.3 Foundation Load-Deformation Characteristics. For linear static and dynamic analysis methods, where foundation flexibility is included in the analysis, the load-deformation behavior of the supporting soil should be represented by an equivalent elastic stiffness using soil properties that are compatible with the soil strain levels associated with the design earthquake motion. The strain-compatible shear modulus, G, and the associated strain-compatible shear wave velocity, v_s, needed for the evaluation of equivalent elastic stiffness are specified in Chapter 19 of the standard or can be based on a site-specific study. Although inclusion of soil flexibility tends to lengthen the fundamental period of the structure, it should not change the maximum period limitations applied when calculating the required base shear of a structure.

A mathematical model incorporating a combined superstructure and foundation system is necessary to assess the effect of foundation and soil deformations on the superstructure elements. Typically, frequency-independent linear springs are included in the mathematical model to represent the load-deformation characteristics of the soil, and the foundation components are either explicitly modeled (e.g., mat foundation supporting a configuration of structural walls) or are assumed to be rigid (e.g., spread footing supporting a column). In specific cases, a spring may be used to model both the soil and foundation component (e.g., grade beams or individual piles).

For dynamic analysis, the standard requires a parametric evaluation with upper and lower bound soil parameters to account for the uncertainty in as-modeled soil stiffness and in situ soil variability and to evaluate the sensitivity of these variations on the superstructure. Sources of uncertainty include variability in the rate of loading, including the cyclic nature of building response, level of strain associated with loading at the design earthquake (or stronger), idealization of potentially nonlinear soil properties as elastic, and variability in the estimated soil properties. To a lesser extent, this variation accounts for variability in the performance of the foundation components, primarily when a rigid foundation is assumed or distribution of cracking of concrete elements is not explicitly modeled.

Commonly used analysis procedures tend to segregate the “structural” components of the foundation (e.g., footing, grade beam, pile, and pile cap) from the supporting (e.g., soil) components. The “structural” components are typically analyzed using standard strength design load combinations and methodologies, whereas the adjacent soil components are analyzed using allowable stress design (ASD) practices, in which earthquake forces (that have been reduced by R) are considered using ASD load combinations, to make comparisons of design forces versus allowable capacities. These “allowable” soil capacities are typically based on expected strength divided by a factor of safety, for a given level of potential deformations.

When design of the superstructure and foundation components is performed using strength-level load combinations, this traditional practice of using allowable stress design to verify soil compliance can become problematic for assessing the behavior of foundation components. The 2009 NEHRP provisions (FEMA 2009a) contain two resource papers (RP 4 and RP 8) that provide guidance on the application of ultimate strength design procedures in the geotechnical design of foundations and the development of foundation load-deformation characterizations for both linear and nonlinear analysis methods. Additional guidance on these topics is contained in ASCE/SEI 41-06 (2007).

C12.13.4 Reduction of Foundation Overturning. Overturning effects at the soil-foundation interface are permitted to be

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reduced by 25% for foundations of structures that satisfy both of the following conditions:

a. The structure is designed in accordance with the equivalent lateral force analysis as set forth in Section 12.8.
b. The structure is not an inverted pendulum or cantilevered column type structure.

Overturning effects at the soil-foundation interface may be reduced by 10% for foundations of structures designed in accordance with the modal analysis requirements of Section 12.9.

C12.13.5 Requirements for Structures Assigned to Seismic Design Category C

C12.13.5.1 Pole-Type Structures. The high contact pressures that develop between an embedded pole and soil as a result of lateral loads make pole-type structures sensitive to earthquake motions. Pole-bending strength and stiffness, the soil lateral bearing capacity, and the permissible deformation at grade level are key considerations in the design. For further discussion of pole-soil interaction, see Section C12.13.6.7.

C12.13.5.2 Foundation Ties. One important aspect of adequate seismic performance is that the foundation system acts as an integral unit, not permitting one column or wall to move appreciably to another. To attain this performance, the standard requires that pile caps be tied together. This requirement is especially important where the use of deep foundations is driven by the existence of soft surface soils.

Multistory buildings often have major columns that run the full height of the building adjacent to smaller columns that support only one level; the calculated tie force should be based on the heavier column load.

The standard permits alternate methods of tying foundations together when appropriate. Relying on lateral soil pressure on pile caps to provide the required restraint is not a recommended method because ground motions are highly dynamic and may occasionally vary between structure support points during a design-level seismic event.

C12.13.5.3 Pile Anchorage Requirements. The pile anchorage requirements are intended to prevent brittle failures of the connection to the pile cap under moderate ground motions. Moderate ground motions can result in pile tension forces or bending moments that could compromise shallow anchorage embedment. Loss of pile anchorage could result in increased structural displacements from rocking, overturning instability, and loss of shearing resistance at the ground surface. A concrete bond to a bare steel pile section usually is unreliable, but connection by means of deformed bars properly developed from the pile cap into concrete confined by a circular pile section is permitted.

C12.13.6 Requirements for Structures Assigned to Seismic Design Categories D through F

C12.13.6.1 Pole-Type Structures. See Section C12.13.5.1.

C12.13.6.2 Foundation Ties. See Section C12.13.5.2. For Seismic Design Categories D through F, the requirement is extended to spread footings on soft soils (Site Class E or F).

C12.13.6.3 General Pile Design Requirement. Design of piles is based on the same response modification coefficient, R, used in design of the superstructure; because inelastic behavior results, piles should be designed with ductility similar to that of the superstructure. When strong ground motions occur, inertial pile-soil interaction may produce plastic hinging in piles near the bottom of the pile cap, and kinematic soil-pile interaction results in bending moments and shearing forces throughout the length of the pile, being higher at interfaces between stiff and soft soil strata. These effects are particularly severe in soft soils and liquefiable soils, so Section 14.2.3.2.1 requires special detailing in areas of concern.

The shears and curvatures in piles caused by inertial and kinematic interaction may exceed the bending capacity of conventionally designed piles, resulting in severe damage. Analysis techniques to evaluate pile bending are discussed by Margason and Holloway (1977) and Mylonakis (2001), and these effects on concrete piles are further discussed by Sheppard (1983). For homogeneous, elastic media and assuming the pile follows the soil, the free-field curvature (soil strains without a pile present) can be estimated by dividing the peak ground acceleration by the square of the shear wave velocity of the soil. Considerable judgment is necessary in using this simple relationship for a layered, inelastic profile with pile-soil interaction effects. Norris (1994) discusses methods to assess pile-soil interaction.

Where determining the extent of special detailing, the designer must consider variation in soil conditions and driven pile lengths, so that adequate ductility is provided at potentially high curvature interfaces. Confinement of concrete piles to provide ductility and maintain functionality of the confined core pile during and after the earthquake may be obtained by use of heavy spiral reinforcement or exterior steel liners.

C12.13.6.4 Batter Piles. Partially embedded batter piles have a history of poor performance in strong ground shaking, as shown by Gerwick and Fotinos (1992). Failure of battered piles has been attributed to design that neglects loading on the piles from ground deformation or assumes that lateral loads are resisted by axial response of piles without regard to moments induced in the pile at the pile cap (Lam and Bertero 1990). Because batter piles are considered to have limited ductility, they must be designed using the load combinations with overstrength factor. Moment-resisting connections between pile and pile cap must resolve the eccentricities inherent in batter pile configurations. This concept is illustrated clearly by EQE Engineering (1991).

C12.13.6.5 Pile Anchorage Requirements. Piles should be anchored to the pile cap to permit energy-dissipating mechanisms, such as pile slip at the pile-soil interface, while maintaining a competent connection. This section of the standard sets forth a capacity design approach to achieve that objective. Anchorages occurring at pile cap corners and edges should be reinforced to preclude local failure of plain concrete sections caused by pile shears, axial loads, and moments.

C12.13.6.6 Splices of Pile Segments. A capacity design approach, similar to that for pile anchorage, is applied to pile splices.

C12.13.6.7 Pile Soil Interaction. Short piles and long slender piles embedded in the earth behave differently when subjected to lateral forces and displacements. The response of a long slender pile depends on its interaction with the soil considering the nonlinear response of the soil. Numerous design aid curves and computer programs are available for this type of analysis, which is necessary to obtain realistic pile moments, forces, and deflections and is common in practice (Ensoft 2004b). More sophisticated models, which also consider inelastic behavior of the pile itself, can be analyzed using general-purpose nonlinear analysis computer programs or closely approximated using the

pile-soil limit state methodology and procedure given by Song et al. (2005).

Short piles (with length-to-diameter ratios no more than 6) can be treated as a rigid body, simplifying the analysis. A method assuming a rigid body and linear soil response for lateral bearing is given in the current building codes. A more accurate and comprehensive approach using this method is given in a study by Czerniak (1957).

C12.13.6.8 Pile Group Effects. The effects of groups of piles, where closely spaced, must be taken into account for vertical and horizontal response. As groups of closely spaced piles move laterally, failure zones for individual piles overlap, and horizontal strength and stiffness response of the pile-soil system is reduced. Reduction factors or "*p*-multipliers" are used to account for these groups of closely spaced piles. For a pile center-to-center spacing of three pile diameters, reduction factors of 0.6 for the leading pile row and 0.4 for the trailing pile rows are recommended by Rollins et al. (1999). Computer programs are available to analyze group effects assuming nonlinear soil and elastic piles (Ensoft 2004a).

C12.14 SIMPLIFIED ALTERNATIVE STRUCTURAL DESIGN CRITERIA FOR SIMPLE BEARING WALL OR BUILDING FRAME SYSTEMS

C12.14.1 General. In recent years, engineers and building officials have become concerned that the seismic design requirements in codes and standards, though intended to make structures perform more reliably, have become so complex and difficult to understand and implement that they may be counterproductive. Because the response of buildings to earthquake ground shaking is complex (especially for irregular structural systems), realistically accounting for these effects can lead to complex requirements. There is a concern that the typical designers of small, simple buildings, which may represent more than 90% of construction in the United States, have difficulty understanding and applying the general seismic requirements of the standard.

The simplified procedure presented in this section of the standard applies to low-rise, stiff buildings. The procedure, which was refined and tested over a five-year period, was developed to be used for a defined set of buildings deemed to be sufficiently regular in structural configuration to allow a reduction of prescriptive requirements. For some design elements, such as foundations and anchorage of nonstructural components, other sections of the standard must be followed, as referenced within Section 12.14.

C12.14.1.1 Simplified Design Procedure. Reasons for the limitations of the simplified design procedure of Section 12.14 are as follows:

1. The procedure was developed to address adequate seismic performance for standard occupancies. Because it was not developed for higher levels of performance associated with structures assigned to Risk Categories III and IV, no importance factor (I_e) is used.
2. Site Class E and F soils require specialized procedures that are beyond the scope of the procedure.
3. The procedure was developed for stiff, low-rise buildings, where higher mode effects are negligible.
4. Only stiff systems where drift is not a controlling design criterion may use the procedure. Because of this limitation, drifts are not computed. The response modification coefficient, R, and the associated system limitations are consistent with those found in the general Chapter 12 requirements.
5. To achieve a balanced design and a reasonable level of redundancy, two lines of resistance are required in each of the two major axis directions. Because of this stipulation, no redundancy factor (ρ) is applied.
6. To reduce the potential for dominant torsional response, at least one line of resistance must be placed on each side of the center of mass.
7. Large overhangs for flexible diaphragm buildings can produce a response that is inconsistent with the assumptions associated with the procedure.
8. A system that satisfies these layout and proportioning requirements avoids torsional irregularity, so calculation of accidental torsional moments is not required.
9. An essentially orthogonal orientation of lines of resistance effectively uncouples response along the two major axis directions, so orthogonal effects may be neglected.
10. Where the simplified design procedure is chosen, it must be used for the entire design, in both major axis directions.
11. Because in-plane and out-of-plane offsets generally create large demands on diaphragms, collectors, and discontinuous elements, which are not addressed by the procedure, these irregularities are prohibited.
12. Buildings that exhibit weak-story behavior violate the assumptions used to develop the procedure.

C12.14.3 Seismic Load Effects and Combinations. The equations for seismic load effects and load combinations in the simplified design procedure are consistent with those for the general procedure, with one notable exception: the overstrength factor (corresponding to Ω_0 in the general procedure) is set at 2.5 for all systems as indicated in Section 12.14.3.2.1. Given the limited systems that can use the simplified design procedure, specifying unique overstrength factors was deemed unnecessary.

C12.14.7 Design and Detailing Requirements. The design and detailing requirements outlined in this section are similar to those for the general procedure. The few differences include the following:

1. Forces used to connect smaller portions of a structure to the remainder of the structures are taken as 0.20 times the short-period design spectral response acceleration, S_{DS}, rather than the general procedure value of 0.133 (Section 12.14.7.1).
2. Anchorage forces for concrete or masonry structural walls for structures with diaphragms that are not flexible are computed using the requirements for nonstructural walls (Section 12.14.7.5).

C12.14.8 Simplified Lateral Force Analysis Procedure

C12.14.8.1 Seismic Base Shear. The seismic base shear in the simplified design procedure, as given by Eq. 12.14-11, is a function of the short-period design spectral response acceleration, S_{DS}. The value for F in the base shear equation addresses changes in dynamic response for buildings that are two or three stories above grade plane (see Section 11.2 for definitions of "grade plane" and "story above grade plane"). As in the general

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procedure (Section 12.8.1.3), S_{DS} may be computed for short, regular structures with S_S taken no greater than 1.5.

C12.14.8.2 Vertical Distribution. The seismic forces for multistory buildings are distributed vertically in proportion to the weight of the respective floor. Given the slightly amplified base shear for multistory buildings, this assumption, along with the limit of three stories above grade plane for use of the procedure, produces results consistent with the more traditional triangular distribution without introducing that more sophisticated approach.

C12.14.8.5 Drift Limits and Building Separation. For the simplified design procedure, which is restricted to stiff shear wall and braced frame buildings, drift need not be calculated. Where drifts are required (such as for structural separations and cladding design) a conservative drift value of 1% is specified.

REFERENCES

American Concrete Institute (ACI). (2008). "Building code requirements for structural concrete and commentary." ACI 318-08.

American Institute of Steel Construction (AISC). (2010). "Seismic provisions for structural steel buildings." AISC 341-10.

American Society of Civil Engineers (ASCE). "Seismic analysis of safety-related nuclear structures and commentary." *ASCE 4-98*, Reston, Va.

American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI). (2007). "Seismic rehabilitation of existing buildings." *ASCE/SEI 41*-06. ASCE, Reston, Va.

Applied Technology Council (ATC). (1978) (Amended 1984). "Tentative provisions for the development of seismic regulations for buildings." *ATC 3-06*. ATC, Redwood City, Calif.

Charney, F. A., and Marshall, J. (2006). "A comparison of the Krawinkler and scissors models for including beam-column joint deformations in the analysis of moment-resisting frames." *AISC Engrg. J.*, 43(1), 31–48.

Chopra, A. K. (2007). *Structural Dynamics*, Prentice Hall, Upper Saddle River, N.J.

Chopra, A. K., and Goel, R. K. (1991). "Evaluation of torsional provisions in seismic codes." *J. Struct. Engrg.*, 117(12), 3762–3782.

Chopra, A. K., and Newmark, N. M. (1980). "Analysis." Chapter 2, *Design of Earthquake Resistant Structures*, E. Rosenblueth, ed., John Wiley and Sons.

Czerniak, E. (1957). "Resistance to overturning of single, short piles." *J. Struct. Div.*, 83(2), 1–25.

De La Llera, J. C., and Chopra, A. K. (1994). "Evaluation of code accidental-torsion provisions from building records." *J. Struct. Engrg.*, 120(2), 597–616.

Earthquake Engineering Research Institute (EERI). (1994). "Northridge Earthquake, January 17, 1994: Preliminary Reconnaissance Report," John F. Hall, ed., EERI, Oakland, Calif., 56–57.

Ensoft, Inc. (2004a). Computer Program GROUP, Version 6.0, A Program for the Analysis of a Group of Piles Subjected to Axial and Lateral Loading, User's Manual and Technical Manual. Ensoft, Austin, Tex.

Ensoft, Inc. (2004b). Computer Program LPILE Plus Version 5.0, A Program for the Analysis of Piles and Drilled Shafts under Lateral Loads, User's Manual and Technical Manual. Ensoft, Austin, Tex.

EQE Engineering. (1991). *Structural Concepts and Details for Seismic Design*, UCRL-CR-106554. Department of Energy, Washington, D.C.

EQE Incorporated. (1983). "The effects of the May 2, 1983, Coalinga, California earthquake on industrial facilities." EQE Inc.

EQE Incorporated. (1985). "Summary of the September 19, 1985, Mexico earthquake." EQE Inc.

EQE Incorporated. (1986a). "Summary of the March 3, 1985, Chile earthquake." EQE Inc.

EQE Incorporated. (1986b). "The effects of the March 3, 1985, Chile earthquake on power and industrial facilities." EQE Inc.

EQE Incorporated. (1986c). "Power and industrial facilities in the epicentral area of the 1985 Mexico earthquake." EQE Inc.

EQE Incorporated. (1987). "Summary of the 1987 Bay of Plenty, New Zealand, earthquake." EQE Inc.

Federal Emergency Management Agency (FEMA). (2009a). "NEHRP recommended seismic provisions for new buildings and other structures." *FEMA P-750*, Building Seismic Safety Council, Federal Emergency Management Agency, Washington, D.C.

Federal Emergency Management Agency (FEMA). (2009b). "Quantification of building seismic performance factors." *FEMA P-695*, Applied Technology Council, Federal Emergency Management Agency, Washington, D.C.

Gerwick, B., Jr., and Fotinos, G. (1992). "Drilled piers and driven piles for foundations in areas of high seismicity." SEAONC Fall Seminar, October 29, San Francisco, Calif.

Goel, R. K., and Chopra, A.K. l. (1997). "Period formulas for moment-resisting frame buildings." *J. Struct. Engrg.*, 123(11), 1454–1461.

Goel, R. K., and Chopra, A. K. (1998). "Period formulas for concrete shear wall buildings." *J. Struct. Engrg.*, 124(4), 426–433.

Gupta, A., and Krawinkler, H. (2000). "Dynamic *P*-delta effects for flexible inelastic steel structures." *J. Struct. Engrg.*, 126(1), 145–154.

Hamburger, R. O., and McCormick, D. L. (2004). "Implications of the January 17, 1994, Northridge earthquake on tilt-up and masonry buildings with wood roofs." *Proceedings, 63rd Annual Convention*, Structural Engineers Association of California, Lake Tahoe, Calif., 243–255.

ICBO International Conference of Building Officials. (1988). *Uniform Building Code*.

Lam, I., and Bertero, V. (1990). "A seismic design of pile foundations for port facilities." *Proceedings of the POLA Seismic Workshop on Seismic Engineering*, March 21–23, San Pedro, California, Port of Los Angeles.

Liew, J. Y. R., Chen, H., and Shanmugam, N. E. (2001). "Inelastic analysis of steel frames with composite beams." *J. Struct. Engrg.*, 127(2), 194–202.

Lopez, O. A., and Cruz, M. (1996). "Number of modes for the seismic design of buildings." *Earthquake Engrg. and Struct. Dyn.*, 25(8), 837–856.

Margason, E., and Holloway, M. (1977). "Pile bending during earthquakes." *Proceedings of the 6th World Conference on Earthquake Engineering*, New Delhi.

Mylonakis, G. (2001). "Seismic pile bending at soil–layer interfaces." *Soils and Found.*, 41(4), 47–58.

Newmark, N. A., and Rosenbleuth, E. (1971). *Fundamentals of Earthquake Engineering*, Prentice Hall.

Norris, G. M. (1994). "Seismic bridge pile foundation behavior." *Proceedings, International Conference on Design and Construction of Deep Foundations*, Federal Highway Administration, Vol. 1.

Paulay, T. (1997). "Are existing seismic torsion provisions achieving design aims?" *Earthquake Spectra*, 13(2), 259–280.

Paulay, T., and Priestley, M. J. N. (1992). *Seismic Design of Reinforced Concrete and Masonry Structures.* John Wiley and Sons.

Rollins, K. M., Peterson, K. T., Weaver, T. J., and Sparks, A. E. (1999). "Static and dynamic lateral load behavior on a full-scale pile group in clay." Brigham Young University, Provo, Utah, and the Utah Department of Transportation, Salt Lake City, June 23.

Rosenblueth, E., and Contreras, H. (1977). "Approximate design for multicomponent earthquakes." *J. Engrg. Mech. Div.*, 103(5), 881–893.

Schaffhausen, R., and Wegmuller, A. (1977). "Multistory rigid frames with composite girders under gravity and lateral forces." *AISC Engrg. J.*, 2nd Quarter.

Sheppard, D. A. 1983. "Seismic design of prestressed concrete piling." *PCI J.*, 28(2), 20–49.

Song, S. T., Chai, Y. H., and Hale, T. H. (2005). "Analytical model for ductility assessment of fixed-head concrete piles." *J. Struct. Engrg.*, 131(7), 1051–1059.

U.S. Nuclear Regulatory Commission. (1999). "Reevaluation of regulatory guidance on modal response combination methods for seismic response spectrum analysis." NUREG/CR-6645, Brookhaven National Laboratory.

Uang, C. M., and Maarouf, A. (1994). "Deflection amplification factor for seismic design provisions." *J. Struct. Engrg.*, 120(8), 2423–2436.

Vamvatsikos, D. (2002). "Seismic performance, capacity and reliability of structures as seen through incremental dynamic analysis." Ph.D. Dissertation, Department of Civil and Environmental Engineering, Stanford University, Palo Alto, Calif.

Wilson, E. L. (2000). "Three-dimensional static and dynamic analysis of structures." Computers and Structures, Inc., Berkeley, Calif.

Wilson, E. L., Der Kiureghian, A., and Bayo, E. P. (1981). "A replacement for the SRSS method in seismic analysis." *Earthquake Engrg. and Struct. Dyn.*, 9(2), 187–194.

OTHER REFERENCES (NOT CITED)

Bernal, D. (1987). "Amplification factors for inelastic dynamic P-delta effects in earthquake analysis." *Earthquake Engrg. and Struct. Dyn.*, 18, 635–681.

Charney, F. A. (1990). "Wind drift serviceability limit state design of multistory buildings." *J. Wind Engrg. and Industrial Aerodynamics*, 36, 203–212.

Degenkolb, H. J. (1987). "A study of the P-delta effect." *Earthquake Spectra*, 3(1).

Griffis, L. (1993). "Serviceability limit states under wind load." *Engrg. J., Am. Inst. Steel Construction*, First Quarter.

Newmark, N. M., and Hall, W. J. (1978). *Development of Criteria for Seismic Review of Selected Nuclear Power Plants*, NUREG/CR-0098, U.S. Nuclear Regulatory Commission.

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CHAPTER C13
SEISMIC DESIGN REQUIREMENTS FOR NONSTRUCTURAL COMPONENTS

C13.1 GENERAL

Chapter 13 defines minimum design criteria for architectural, mechanical, electrical, and other nonstructural systems and components, recognizing structure use, occupant load, the need for operational continuity, and the interrelation of structural, architectural, mechanical, electrical, and other nonstructural components. Nonstructural components are designed for design earthquake ground motions as defined in Section 11.2 and determined in Section 11.4.4 of the standard. In contrast to structures, which are implicitly designed for a low probability of collapse when subjected to the maximum considered earthquake (MCE) ground motions, there are no implicit performance goals associated with the MCE for nonstructural components. Performance goals associated with the design earthquake are discussed in Section C13.1.3.

Suspended or attached nonstructural components that could detach either in full or in part from the structure during an earthquake are referred to as falling hazards and may represent a serious threat to property and life safety. Critical attributes that influence the hazards posed by these components include their weight, their attachment to the structure, their failure or breakage characteristics (e.g., nonshatterproof glass), and their location relative to occupied areas (e.g., over an entry or exit, a public walkway, an atrium, or a lower adjacent structure). Architectural components that pose potential falling hazards include parapets, cornices, canopies, marquees, glass, large ornamental elements (e.g., chandeliers), and building cladding. In addition, suspended mechanical and electrical components (e.g., mixing boxes, piping, and ductwork) may represent serious falling hazards. Figures C13.1-1 through C13.1-4 show damage to nonstructural components in past earthquakes.

Components whose collapse during an earthquake could result in blockage of the means of egress deserve special consideration. The term "means of egress" is used commonly in building codes with respect to fire hazard. Egress paths may include intervening aisles, doors, doorways, gates, corridors, exterior exit balconies, ramps, stairways, pressurized enclosures, horizontal exits, exit passageways, exit courts, and yards. Items whose failure could jeopardize the means of egress include walls around stairs and corridors, veneers, cornices, canopies, heavy partition systems, ceilings, architectural soffits, light fixtures, and other ornaments above building exits or near fire escapes. Examples of components that generally do not pose a significant falling hazard include fabric awnings and canopies. Architectural, mechanical, and electrical components that, if separated from the structure, fall in areas that are not accessible to the public (e.g., into a mechanical shaft or light well) also pose little risk to egress routes.

For some architectural components, such as exterior cladding elements, wind design forces may exceed the calculated seismic design forces. Nevertheless, seismic detailing requirements may still govern the overall structural design. Where this is a possibility, it must be investigated early in the structural design process.

The seismic design of nonstructural components may involve consideration of nonseismic requirements that are affected by seismic bracing. For example, accommodation of thermal expansion in pressure piping systems often is a critical design consideration, and seismic bracing for these systems must be arranged in a manner that accommodates thermal movements. Particularly in the case of mechanical and electrical systems, the design for seismic loads should not compromise the functionality, durability, or safety of the overall design; this requires collaboration among the various disciplines of the design and construction team.

For various reasons (e.g., business continuity), it may be desirable to consider higher performance than that required by the building code. For example, to achieve continued operability of a piping system, it is necessary to prevent unintended operation of valves or other inline components in addition to preventing collapse and providing leak tightness. Higher performance also is required for components containing substantial quantities of hazardous contents (as defined in Section 11.2). These components must be designed to prevent uncontrolled release of those materials.

The requirements of Chapter 13 are intended to apply to new construction and tenant improvements installed at any time during the life of the structure, provided they are listed in Table 13.5-1 or 13.6-1. Furthermore, they are intended to reduce (not eliminate) the risk to occupants and to improve the likelihood that essential facilities remain functional. Although property protection (in the sense of investment preservation) is a possible consequence of implementation of the standard, it is not currently a stated or implied goal; a higher level of protection may be advisable if such protection is desired or required.

C13.1.1 Scope. The requirements for seismic design of nonstructural components apply to the nonstructural component and to its supports and attachments to the main structure. In some cases, as defined in Section 13.2, it is necessary to consider explicitly the performance characteristics of the component. The requirements are intended to apply only to permanently attached components, not to furniture, temporary items, or mobile units. Furniture, such as tables, chairs, and desks, may shift during strong ground shaking but generally poses minimal hazards provided that it does not obstruct emergency egress routes. Storage cabinets, tall bookshelves, and other items of significant mass do not fall into this category and should be anchored or braced in accordance with this chapter.

Temporary items are those that remain in place for short periods of time (months, not years). Components that are

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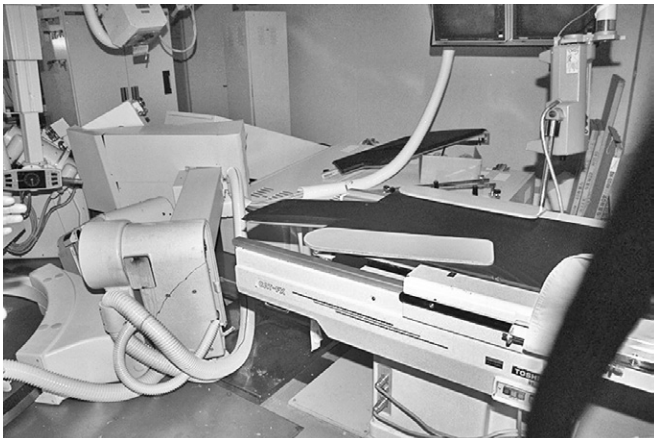

FIGURE C13.1-1 Hospital Imaging Equipment that Fell from Overhead Mounts

FIGURE C13.1-2 Collapsed Light Fixtures

FIGURE C13.1-3 Collapsed Duct and HVAC Diffuser

FIGURE C13.1-4 Damaged Ceiling System

FIGURE C13.1-5 Toppled Storage Cabinets

expected to remain in place for periods of a year or longer, even if they are designed to be movable, should be considered permanent for the purposes of this section. Modular office systems are considered permanent because they ordinarily remain in place for long periods. In addition, they often include storage units that have significant capacity and may topple in an earthquake. They are subject to the provisions of Section 13.5.8 for partitions if they exceed 6 ft in height. Mobile units include components that are moved from one point in the structure to another during ordinary use. Examples include desktop computers, office equipment, and other components that are not permanently attached to the building utility systems (Fig. C13.1-5). Components that are mounted on wheels to facilitate periodic maintenance or cleaning but that otherwise remain in the same location (e.g., server racks) are not considered movable for the purposes of anchorage and bracing. Likewise, skid-mounted components (as shown in Fig. C13.1-6), as well as the skids themselves, are considered permanent equipment.

In all cases, equipment must be anchored if it is permanently attached to utility services (electricity, gas, and water). For the purposes of this requirement, "permanently attached" should be understood to include all electrical connections except NEMA 5-15 and 5-20 straight-blade connectors (duplex receptacles).

C13.1.2 Seismic Design Category. The requirements for nonstructural components are based in part on the seismic design category (SDC) to which they are assigned. As the SDC is established considering factors not unique to specific nonstructural components, all nonstructural components occupying or attached to a structure are assigned to the same SDC as the structure.

C13.1.3 Component Importance Factor. Performance expectations for nonstructural components often are defined in terms

FIGURE C13.1-6 Skid-mounted components.

of the functional requirements of the structure to which the components are attached. Although specific performance goals for nonstructural components have yet to be defined in building codes, the component importance factor (I_p) implies performance levels for specific cases. For noncritical nonstructural components (those with an importance factor, I_p, of 1.0), the following behaviors are anticipated for shaking of different levels of intensity:

1. Minor earthquake ground motions—minimal damage; not likely to affect functionality;
2. Moderate earthquake ground motions—some damage that may affect functionality; and
3. Design earthquake ground motions—major damage but significant falling hazards are avoided; likely loss of functionality.

Components with importance factors greater than 1.0 are expected to remain in place, sustain limited damage, and when necessary, function after an earthquake (see Section C13.2.2). These components can be located in structures that are not assigned to Risk Category IV. For example, fire sprinkler piping systems have an importance factor, I_p, of 1.5 in all structures because these essential systems should function after an earthquake. Egress stairways are assigned an I_p of 1.5 as well, although in many cases the design of these stairways is dictated by differential displacements, not inertial force demands.

The component importance factor is intended to represent the greater of the life-safety importance of the component and the hazard-exposure importance of the structure. It indirectly influences the survivability of the component via required design forces and displacement levels, as well as component attachments and detailing. Although this approach provides some degree of confidence in the seismic performance of a component, it may not be sufficient in all cases. For example, individual ceiling tiles may fall from a ceiling grid that has been designed for larger forces. This may not represent a serious falling hazard if the ceiling tiles are made of lightweight materials, but it may lead to blockage of critical egress paths or disruption of the facility function. When higher levels of confidence in performance are required, the component is classified as a designated seismic system (Section 11.2), and in certain cases, seismic qualification of the component or system is necessary. Seismic qualification approaches are provided in Sections 13.2.5 and 13.2.6. In addition, seismic qualification approaches presently in use by the Department of Energy (DOE) can be applied.

Risk Category IV structures are intended to be functional after a design earthquake; critical nonstructural components and equipment in such structures are designed with I_p equal to 1.5. This requirement applies to most components and equipment because damage to vulnerable unbraced systems or equipment may disrupt operations after an earthquake even if they are not directly classified as essential to life safety. The nonessential and nonhazardous components are themselves not affected by this requirement. Instead, requirements focus on the supports and attachments. UFC 3-310-04 (DOD 2007) has additional guidance for improved performance.

C13.1.4 Exemptions. Several classes of nonstructural components are exempted from the requirements of Chapter 13. The exemptions are made on the assumption that, either due to their inherent strength and stability or the lower level of earthquake demand (accelerations and relative displacements), or both, these nonstructural components and systems can achieve the performance goals described earlier in this commentary without explicitly satisfying the requirements of this chapter.

The requirements are intended to apply only to permanent components, not furniture and temporary or mobile equipment. Furniture (with the exception of more massive elements like storage cabinets) may shift during strong ground shaking but poses minimal hazards. Equipment must be anchored if it is permanently attached to the structure utility services, such as electricity, gas, or water. For the purposes of this requirement, "permanently attached" includes all electrical connections except plugs for duplex receptacles. See Section 13.1.1 for a discussion of temporary components.

Mobile units include components that are moved from one point in the structure to another during ordinary use. Examples include desktop computers, office equipment, and other components that are not permanently attached to the building utility systems. Components mounted on wheels to facilitate periodic maintenance or cleaning but that otherwise remain in the same location are not considered movable for the purposes of anchorage and bracing.

Furniture resting on floors, such as tables, chairs, and desks, may shift during strong ground shaking, but they generally pose minimal hazards, provided they do not obstruct emergency egress routes. Examples also include desktop computers, office equipment, and other components that are not permanently attached to the building utility systems.

With the exception of parapets supported by bearing walls or shear walls, all components in Seismic Design Categories A and B are exempt because of the low levels of ground shaking expected. Parapets are not exempt because experience has shown that these items can fail and pose a significant falling hazard, even at low-level shaking levels.

The exemption for mechanical and electrical components in Seismic Design Categories D, E, or F based on weight and location of the center of mass is particularly applicable to vertical equipment racks and similar components. Where detailed information regarding the center of mass of the intended installation is unavailable, a conservative estimate based on potential equipment configurations should be used.

Although the exemptions listed in Section 13.1.4 are intended to waive bracing requirements for nonstructural components that are judged to pose negligible life safety hazard, in some cases it may nevertheless be advisable to consider bracing (in consultation with the owner) for exempted components to minimize

repair costs and/or disproportionate loss (e.g., art works of high value).

C13.1.5 Application of Nonstructural Component Requirements to Nonbuilding Structures. At times, a nonstructural component should be treated as a nonbuilding structure. When the physical characteristics associated with a given class of nonstructural components vary widely, judgment is needed to select the appropriate design procedure and coefficients. For example, cooling towers vary from small packaged units with an operating weight of 2,000 lb or less to structures the size of buildings. Consequently, design coefficients for the design of "cooling towers" are found both in Tables 13.6-1 and 15.4-2. Small cooling towers are best designed as nonstructural components using the provisions of Chapter 13, whereas large ones are clearly nonbuilding structures that are more appropriately designed using the provisions of Chapter 15. Similar issues arise for other classes of nonstructural component (e.g., boilers and bins). Guidance on determining whether an item should be treated as a nonbuilding structure or nonstructural component for the purpose of seismic design is provided in Bachman and Dowty (2008).

Premanufactured modular mechanical units are considered nonbuilding structures supporting nonstructural components. The entire system (all modules assembled) can be shake-table qualified or qualified separately as subsystems. Modular mechanical units house various nonstructural components that are subject to all the design requirements of Chapter 13.

The specified weight limit for nonstructural components (25% relative to the combined weight of the structure and component) relates to the condition at which dynamic interaction between the component and the supporting structural system is potentially significant. Section 15.3.2 contains requirements for addressing this interaction in design.

C13.1.6 Reference Documents. Professional and trade organizations have developed nationally recognized codes and standards for the design and construction of specific mechanical and electrical components. These documents provide design guidance for normal and upset (abnormal) operating conditions and for various environmental conditions. Some of these documents include earthquake design requirements in the context of the overall mechanical or electrical design. It is the intent of the standard that seismic requirements in referenced documents be used. The developers of these documents are familiar with the expected performance and failure modes of the components; however, the documents may be based on design considerations not immediately obvious to a structural design professional. For example, in the design of industrial piping, stresses caused by seismic inertia forces typically are not added to those caused by thermal expansion.

Where reference documents have been adopted specifically by this standard as meeting the force and displacement requirements of this chapter with or without modification, they are considered to be a part of the standard.

There is a potential for misunderstanding and misapplication of reference documents for the design of mechanical and electrical systems. A registered design professional familiar with both the standard and the reference documents used should be involved in the review and acceptance of the seismic design.

Even when reference documents for nonstructural components lack specific earthquake design requirements, mechanical and electrical equipment constructed in accordance with industry-standard reference documents have performed well historically when properly anchored. Nevertheless, manufacturers of mechanical and electrical equipment are expected to consider seismic loads in the design of the equipment itself, even when such consideration is not explicitly required by this chapter.

Although some reference documents provide requirements for seismic capacity appropriate to the component being designed, the seismic demands used in design may not be less than those specified in the standard.

Specific guidance for selected mechanical and electrical components and conditions is provided in Section 13.6.

Unless exempted in Section 13.1.4, components should be anchored to the structure and to promote coordination required supports and attachments should be detailed in the construction documents. Reference documents may contain explicit instruction for anchorage of nonstructural components. The anchorage requirements of Section 13.4 must be satisfied in all cases, however, to ensure a consistent level of robustness in the attachments to the structure.

C13.1.7 Reference Documents Using Allowable Stress Design. Many nonstructural components are designed using specifically developed reference documents that are based on allowable stress loads and load combinations and generally permit increases in allowable stresses for seismic loading. Although Section 2.4.1 of the standard does not permit increases in allowable stresses, Section 13.1.7 explicitly defines the conditions for stress increases in the design of nonstructural components where reference documents provide a basis for earthquake-resistant design.

C13.2 GENERAL DESIGN REQUIREMENTS

C13.2.1 Applicable Requirements for Architectural, Mechanical, and Electrical Components, Supports, and Attachments. Compliance with the requirements of Chapter 13 may be accomplished by project-specific design or by a manufacturer's certification of seismic qualification of a system or component. When compliance is by manufacturer's certification, the items must be installed in accordance with the manufacturer's requirements. Evidence of compliance may be provided in the form of a signed statement from a representative of the manufacturer or from the registered design professional indicating that the component or system is seismically qualified. One or more of the following options for evidence of compliance may be applicable:

1. An analysis (e.g., of a distributed system such as piping) that includes derivation of the forces used for the design of the system, the derivation of displacements and reactions, and the design of the supports and anchorages;
2. A test report, including the testing configuration and boundary conditions used (where testing is intended to address a class of components, the range of items covered by the testing performed should also include the justification of similarities of the items that make this certification valid); and/or
3. An experience data report.

Components addressed by the standard include individual simple units and assemblies of simple units for which reference documents establish seismic analysis or qualification requirements. Also addressed by the standard are complex architectural, mechanical, and electrical systems for which reference documents either do not exist or exist for only elements of the system. In the design and analysis of both simple components and complex systems, the concepts of flexibility and ruggedness often can assist the designer in determining the necessity for analysis and, when analysis is necessary, the extent and methods

by which seismic adequacy may be determined. These concepts are discussed in Section C13.6.1.

C13.2.2 Special Certification Requirements for Designated Seismic Systems. This section addresses the qualification of active designated seismic equipment, its supports, and attachments with the goals of improving survivability and achieving a high level of confidence that a facility will be functional after a design earthquake. Where components are interconnected, the qualification should provide the permissible forces (e.g., nozzle loads) and, as applicable, anticipated displacements of the component at the connection points to facilitate assessment for consequential damage, in accordance with Section 13.2.3. Active equipment has parts that rotate, move mechanically, or are energized during operation. Active designated seismic equipment constitutes a limited subset of designated seismic systems. Failure of active designated seismic equipment itself may pose a significant hazard. For active designated seismic equipment, failure of structural integrity and loss of function are to be avoided.

Examples of active designated seismic equipment include mechanical (HVAC and refrigeration) or electrical (power supply distribution) equipment, medical equipment, fire pump equipment, and uninterruptible power supplies for hospitals.

There are practical limits on the size of a component that can be qualified via shake-table testing. Components too large to be qualified by shake-table testing need to be qualified by a combination of structural analysis and qualification testing or empirical evaluation through a subsystem approach. Subsystems of large, complex components (e.g., very large chillers or skid-mounted equipment assemblies) can be qualified individually, and the overall structural frame of the component can be evaluated by structural analysis.

Evaluating postearthquake operational performance for active equipment by analysis generally involves sophisticated modeling with experimental validation and may not be reliable. Therefore, the use of analysis for active or energized components is not permitted unless a comparison can be made to components that have been otherwise deemed as rugged. As an example, a transformer is energized but contains components that can be shown to remain linearly elastic and are inherently rugged. However, switch equipment that contains fragile components is similarly energized but not inherently rugged, and it therefore cannot be certified solely by analysis. For complex components, testing or experience may therefore be the only practical way to ensure that the equipment will be operable after a design earthquake. Past earthquake experience has shown that most active equipment is inherently rugged. Therefore, evaluation of experience data, together with analysis of anchorage, is adequate to demonstrate compliance of active equipment such as pumps, compressors, and electric motors. In other cases, such as for motor control centers and switching equipment, shake-table testing may be required.

With some exceptions (e.g., elevator motors), experience indicates that active mechanical and electrical components that contains electric motors of more than 10 hp or that have a thermal exchange capacity greater than 200 MBH are unlikely to merit the exemption from shake-table testing on the basis of inherent ruggedness. Components with lesser motor horsepower and thermal exchange capacity are generally considered to be small active components and are deemed rugged. Exceptions to this rule may be appropriate for specific cases, such as elevator motors that have higher horsepower but have been shown by experience to be rugged. Analysis is still required to ensure the structural integrity of the nonactive components. For example, a 15-ton condenser would require analysis of the load path between the condenser fan and coil to the building structure attachment.

C13.2.3 Consequential Damage. Although the components identified in Tables 13.5-1 and 13.6-1 are listed separately, significant interrelationships exist and must be considered. Consequential damage occurs because of interaction between components and systems. Even "braced" components displace, and the displacement between lateral supports can be significant in the case of distributed systems such as piping systems, cable and conduit systems, and other linear systems. It is the intent of the standard that the seismic displacements considered include both relative displacement between multiple points of support (addressed in Section 13.3.2) and, for mechanical and electrical components, displacement within the component assemblies. Impact of components must be avoided, unless the components are fabricated of ductile materials that have been shown to be capable of accommodating the expected impact loads. With protective coverings, ductile mechanical and electrical components and many more fragile components are expected to survive all but the most severe impact loads. Flexibility and ductility of the connections between distribution systems and the equipment to which they attach is essential to the seismic performance of the system.

The determination of the displacements that generate these interactions are not addressed explicitly in Section 13.3.2.1. That section concerns relative displacement of support points. Consequential damage may occur because of displacement of components and systems between support points. For example, in older suspended ceiling installations, excessive lateral displacement of a ceiling system may fracture sprinkler heads that project through the ceiling. A similar situation may arise if sprinkler heads projecting from a small-diameter branch line pass through a rigid ceiling system. Although the branch line may be properly restrained, it may still displace sufficiently between lateral support points to affect other components or systems. Similar interactions occur where a relatively flexible distributed system connects to a braced or rigid component.

The potential for impact between components that are in contact with or close to other structural or nonstructural components must be considered. However, where considering these potential interactions, the designer must determine if the potential interaction is both credible and significant. For example, the fall of a ceiling panel located above a motor control center is a credible interaction because the falling panel in older suspended ceiling installations can reach and impact the motor control center. An interaction is significant if it can result in damage to the target. Impact of a ceiling panel on a motor control center may not be significant because of the light weight of the ceiling panel. Special design consideration is appropriate where the failure of a nonstructural element could adversely influence the performance of an adjacent critical nonstructural component, such as an emergency generator.

C13.2.4 Flexibility. In many cases, flexibility is more important than strength in the performance of distributed systems, such as piping and ductwork. A good understanding of the displacement demand on the system, as well as its displacement capacity, is required. Components or their supports and attachments must be flexible enough to accommodate the full range of expected differential movements; some localized inelasticity is permitted in accommodating the movements. Relative movements in all directions must be considered. For example, even a braced branch line of a piping system may displace, so it needs to be connected to other braced or rigid components in a manner that accommodates the displacements without failure (Fig. C13.2-1).

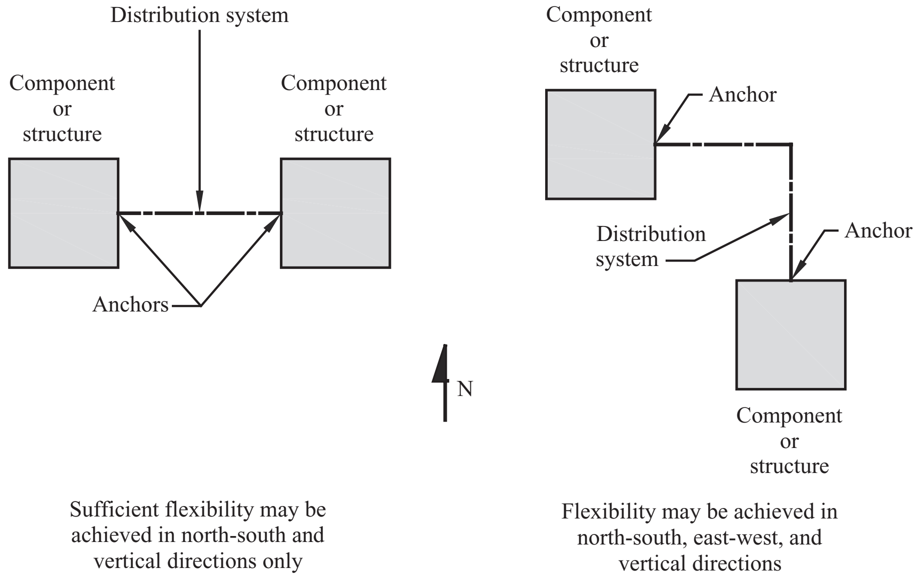

FIGURE C13.2-1 Schematic Plans Illustrating Branch Line Flexibility

A further example is provided by cladding units (such as precast concrete wall units). Often very rigid in plane, cladding units require connections capable of accommodating story drift if attached at more than one level. (See Fig. C13.3-3 for an illustration.)

If component analysis assumes rigid anchors or supports, the predicted loads and local stresses can be unrealistically large, so it may be necessary to consider anchor and/or support stiffness.

C13.2.5 Testing Alternative for Seismic Capacity Determination. Testing is a well-established alternative method of seismic qualification for small- to medium-size equipment. Several national reference documents have testing requirements adaptable for seismic qualification. One such reference document (ICC-ES AC156) is a shake-table testing protocol that has been adopted by the International Code Council Evaluation Service. It was developed specifically to be consistent with acceleration demands (that is, force requirements) of the standard.

The development or selection of testing and qualification protocols should at a minimum include the following:

1. Description of how the protocol meets the intent for the project-specific requirements and relevant interpretations of the standard;
2. Definition of a test input motion with a response spectrum that meets or exceeds the design earthquake spectrum for the site;
3. Accounting for dynamic amplification caused by above-grade equipment installations (consideration of the actual dynamic characteristics of the primary support structure is permitted, but not required);
4. Definition of how shake-table input demands were derived;
5. Definition and establishment of a verifiable pass/fail acceptance criterion for the seismic qualification based on the equipment importance factor and consistent with the building code and project-specific design intent; and
6. Development of criteria that can be used to rationalize test unit configuration requirements for highly variable equipment product lines.

To aid the design professional in assessing the adequacy of the manufacturer's certificate of compliance, it is recommended that certificates of compliance include the following:

1. Product family or group covered;
2. Building code(s) and standard(s) for which compliance was evaluated;
3. Testing standard used;
4. Performance objective and corresponding importance factor ($I_p = 1.0$ or $I_p = 1.5$);
5. Seismic demand for which the component is certified, including code and/or standard design parameters used to calculate seismic demand (such as values used for a_p, R_p, and site class); and
6. Installation restrictions, if any (grade, floor, or roof level).

Without a test protocol recognized by the building code, qualification testing is inconsistent and difficult to verify. The use of ICC-ES AC156 simplifies the task of compliance verification because it was developed to address directly the testing alternative for nonstructural components, as specified in the standard. It also sets forth minimum test plan and report deliverables.

Use of other standards or ad hoc protocols to verify compliance of nonstructural components with the requirement of the standard should be considered carefully and used only where project-specific requirements cannot be met otherwise.

Where other qualification test standards are used, in whole or in part, it is necessary to verify compliance with this standard. For example, IEEE 693 indicates that it is to be used for the sole purpose of qualifying electrical equipment (specifically listed in the standard) for use in utility substations. Where equipment testing has been conducted to other standards (for instance, testing done in compliance with IEEE 693), a straightforward approach would be to permit evaluation, by the manufacturer, of the test plan and data to validate compliance with the requirements of ICC-ES AC156, because it was developed specifically to comply with the seismic demands of this standard.

The qualification of mechanical and electrical components for seismic loads alone may not be sufficient to achieve high-performance objectives. Establishing a high confidence that performance goals will be met requires consideration of the

performance of structures, systems (e.g., fluid, mechanical, electrical, and instrumentation), and their interactions (e.g., interaction of seismic and other loads), as well as compliance with installation requirements.

C13.2.6 Experience Data Alternative for Seismic Capacity Determination. An established method of seismic qualification for certain types of nonstructural components is the assessment of data for the performance of similar components in past earthquakes. The seismic capacity of the component in question is extrapolated based on estimates of the demands (e.g., force or displacement) to which the components in the database were subjected. Procedures for such qualification have been developed for use in nuclear facility applications by the Seismic Qualification Utility Group (SQUG) of the Electric Power Research Institute.

The SQUG rules for implementing the use of experience data are described in a proprietary Generic Implementation Procedure database. It is a collection of findings from detailed engineering studies by experts for equipment from a variety of utility and industrial facilities.

Valid use of experience data requires satisfaction of rules that address physical characteristics; manufacturer's classification and standards; and findings from testing, analysis, and expert consensus opinion.

Four criteria are used to establish seismic qualification by experience, as follows:

1. Seismic capacity versus demand (a comparison with a bounding spectrum);
2. Earthquake experience database cautions and inclusion rules;
3. Evaluation of anchorage; and
4. Evaluation of seismic interaction.

Experience data should be used with care, because the design and manufacture of components may have changed considerably in the intervening years. The use of this procedure is also limited by the relative rarity of strong-motion instrument records associated with corresponding equipment experience data.

C13.2.7 Construction Documents. Where the standard requires seismic design of components or their supports and attachments, appropriate construction documents defining the required construction and installation must be prepared. These documents facilitate the special inspection and testing needed to provide a reasonable level of quality assurance. Of particular concern are large nonstructural components (such as rooftop chillers) whose manufacture and installation involve multiple trades and suppliers, and which impose significant loads on the supporting structure. In these cases, it is important that the construction documents used by the various trades and suppliers be prepared by a registered design professional to satisfy the seismic design requirements.

The information required to prepare construction documents for component installation includes the dimensions of the component, the locations of attachment points, the operating weight, and the location of the center of mass. For instance, if an anchorage angle is attached to the side of a metal chassis, the gauge and material of the chassis must be known so that the number and size of required fasteners can be determined. Or when a piece of equipment has a base plate that is anchored to a concrete slab with expansion anchors, the drawings must show the base plate's material and thickness, the diameter of the bolt holes in the plate, and the size and depth of embedment of the anchor bolts. If the plate is elevated above the slab for leveling, the construction documents must also show the maximum gap permitted between the plate and the slab.

C13.3 SEISMIC DEMANDS ON NONSTRUCTURAL COMPONENTS

The seismic demands on nonstructural components, as defined in this section, are acceleration demands and relative displacement demands. Acceleration demands are represented by equivalent static forces. Relative displacement demands are provided directly and are based on either the actual displacements computed for the structure or the maximum allowable drifts that are permitted for the structure.

C13.3.1 Seismic Design Force. The seismic design force for a component depends on the weight of the component, the component importance factor, the component response modification factor, the component amplification factor, and the component acceleration at a point of attachment to the structure. The forces prescribed in this section of the standard reflect the dynamic and structural characteristics of nonstructural components. As a result of these characteristics, forces used for verification of component integrity and design of connections to the supporting structure typically are larger than those used for design of the overall seismic force-resisting system.

Certain nonstructural components lack the desirable attributes of structures (such as ductility, toughness, and redundancy) that permit the use of greatly reduced lateral design forces. Thus values for the response modification factor, R_p, in Tables 13.5-1 and 13.6-1 generally are smaller than R values for structures. These R_p values, used to represent the energy absorption capability of a component and its attachments, depend on both overstrength and deformability. At present, these potentially separate considerations are combined in a single factor. The tabulated values are based on the collective judgment of the responsible committee.

Beginning with the 2005 edition of ASCE 7, significant adjustments have been made to tabulated R_p values for certain mechanical and electrical systems. For example, the value of R_p for welded steel piping systems is increased from 3.5 to 9. The a_p value increased from 1.0 to 2.5, so although it might appear that forces on such piping systems have been reduced greatly, the net change is negligible because R_p/a_p changes from 3.5 to 3.6. The minimum seismic design force of Eq. 13.3-3, which governs in many cases, is unchanged.

The component amplification factor (a_p) represents the dynamic amplification of component responses as a function of the fundamental periods of the structure (T) and component (T_p). When components are designed or selected, the structural fundamental period is not always defined or readily available. The component fundamental period (T_p) is usually only accurately obtained by shake-table or pull-back tests and is not available for the majority of components. Tabulated a_p values are based on component behavior that is assumed to be either rigid or flexible. Where the fundamental period of the component is less than 0.06 s, dynamic amplification is not expected and the component is considered rigid. The tabulation of assumed a_p values is not meant to preclude more precise determination of the component amplification factor where the fundamental periods of both structure and component are available. The National Center for Earthquake Engineering Research formulation shown in Fig. C13.3-1 may be used to compute a_p as a function of T_p/T.

Dynamic amplification occurs where the period of a nonstructural component closely matches that of any mode of the supporting structure, although this effect may not be significant

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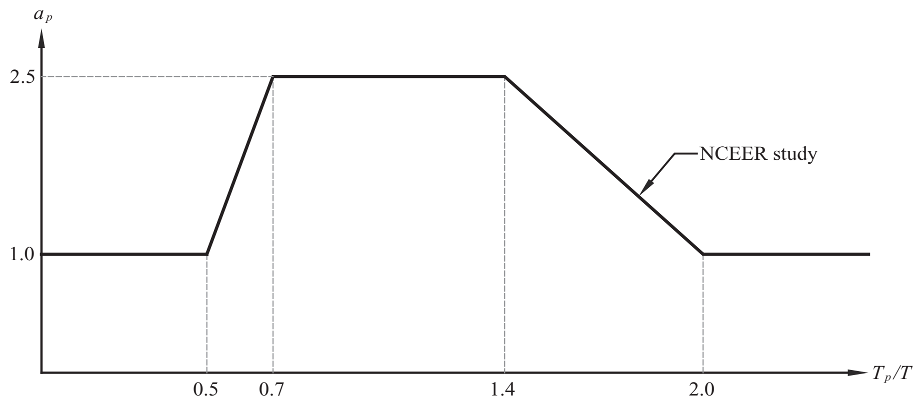

FIGURE C13.3-1 NCEER Formulation for a_p as Function of Structural and Component Periods

depending on the ground motion. For most buildings, the primary mode of vibration in each direction has the most influence on the dynamic amplification for nonstructural components. For long-period structures (such as tall buildings), where the period of vibration of the fundamental mode is greater than 3.5 times T_s, higher modes of vibration may have periods that more closely match the period of nonstructural components. For this case, it is recommended that amplification be considered using such higher mode periods in lieu of the higher fundamental period. This approach may be generalized by computing floor response spectra for various levels that reflect the dynamic characteristics of the supporting structure to determine how amplification varies as a function of component period. Calculation of floor response spectra can be complex, but simplified procedures are presented in Kehoe and Hachem (2003). Consideration of nonlinear behavior of the structure greatly complicates the analysis.

Equation 13.3-1 represents a trapezoidal distribution of floor accelerations within a structure, varying linearly from the acceleration at the ground (taken as 0.4 S_{DS}) to the acceleration at the roof (taken as 1.2 S_{DS}). The ground acceleration (0.4 S_{DS}) is intended to be the same acceleration used as design input for the structure itself, including site effects. The roof acceleration is established as three times the input ground acceleration based on examination of recorded in-structure acceleration data for short and moderate height structures in response to large California earthquakes. Work by Miranda and Singh suggests that, for taller structures, the amplification with height may vary significantly because of higher mode effects. Where more information is available, Eq. 13.3-4 permits an alternate determination of the component design forces based on the dynamic properties of the structure.

Equation 13.3-3 establishes a minimum seismic design force, F_p, that is consistent with current practice. Equation 13.3-2 provides a simple maximum value of F_p that prevents multiplication of the individual factors from producing a design force that would be unreasonably high, considering the expected nonlinear response of support and component. Figure C13.3-2 illustrates the distribution of the specified lateral design forces.

For elements with points of attachment at more than one height, it is recommended that design be based on the average of values of F_p determined individually at each point of attachment (but with the entire component weight, W_p) using Eqs. 13.3-1 through 13.3-3.

Alternatively, for each point of attachment, a force F_p may be determined using Eqs. 13.3-1 through 13.3-3, with the portion of the component weight, W_p, tributary to the point of attachment. For design of the component, the attachment force F_p must

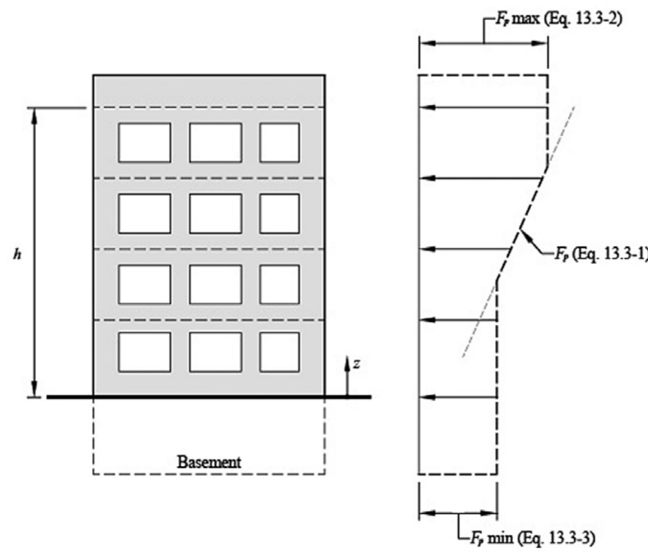

FIGURE C13.3-2 Lateral Force Magnitude over Height

be distributed relative to the component's mass distribution over the area used to establish the tributary weight. To illustrate these options, consider a solid exterior nonstructural wall panel, supported top and bottom, for a one-story building with a rigid diaphragm. The values of F_p computed, respectively, for the top and bottom attachments using Eqs. 13.3-1 through 13.3-3 are 0.48 $S_{DS}I_pW_p$ and 0.30 $S_{DS}I_pW_p$. In the recommended method, a uniform load is applied to the entire panel based on 0.39 $S_{DS}I_pW_p$. In the alternative method, a trapezoidal load varying from 0.48 $S_{DS}I_pW_p$ at the top to 0.30 $S_{DS}I_pW_p$ at the bottom is applied. Each anchorage force is then determined considering static equilibrium of the complete component subject to all the distributed loads.

Cantilever parapets that are part of a continuous element should be checked separately for parapet forces. The seismic force on any component must be applied at the center of gravity of the component and must be assumed to act in any horizontal direction. Vertical forces on nonstructural components equal to ± 0.2 $S_{DS}W_p$ are specified in Section 13.3.1 and are intended to be applied to all nonstructural components and not just cantilevered elements. Nonstructural concrete or masonry walls laterally supported by flexible diaphragms must be anchored out of plane in accordance with Section 12.11.2.

C13.3.2 Seismic Relative Displacements. The equations of this section are for use in design of cladding, stairways, windows, piping systems, sprinkler components, and other components connected to one structure at multiple levels or to multiple structures. Two equations are given for each situation. Equations 13.3-6 and 13.3-8 produce structural displacements as determined by elastic analysis, unreduced by the structural response modification factor (R). Because the actual displacements may not be known when a component is designed or procured, Eqs. 13.3-7 and 13.3-9 provide upper-bound displacements based on structural drift limits. Use of upper-bound equations may facilitate timely design and procurement of components but may also result in costly added conservatism.

The value of seismic relative displacements is taken as the calculated displacement, D_p, times the importance factor, I_e, because the elastic displacement calculated in accordance with Eq. 12.8-15 to establish δ_x (and thus D_p) is adjusted for I_e in keeping with the philosophy of displacement demand for the structure. For component design, the unreduced elastic displacement is appropriate.

The standard does not provide explicit acceptance criteria for the effects of seismic relative displacements, except for glazing. Damage to nonstructural components caused by relative displacement is acceptable, provided the performance goals defined elsewhere in the chapter are achieved.

The design of some nonstructural components that span vertically in the structure can be complicated when supports for the element do not occur at horizontal diaphragms. The language in Section 13.3.2 was previously amended to clarify that story drift must be accommodated in the elements that actually distort. For example, a glazing system supported by precast concrete spandrels must be designed to accommodate the full story drift, even though the height of the glazing system is only a fraction of the floor-to-floor height. This condition arises because the precast spandrels behave as rigid bodies relative to the glazing system and therefore all the drift must be accommodated by anchorage of the glazing unit, the joint between the precast spandrel and the glazing unit, or some combination of the two.

C13.3.2.1 Displacements within Structures. Seismic relative displacements can subject components or systems to unacceptable stresses. The potential for interaction resulting from component displacements (in particular for distributed systems) and the resulting impact effects should also be considered (see Section 13.2.3).

These interrelationships may govern the clearance requirements between components or between components and the surrounding structure. Where sufficient clearance cannot be provided, consideration should be given to the ductility and strength of the components and associated supports and attachments to accommodate the potential impact.

Where nonstructural components are supported between, rather than at, structural levels, as frequently occurs for glazing systems, partitions, stairs, veneers, and mechanical and electrical distributed systems, the height over which the displacement demand, D_p, must be accommodated may be less than the story height, h_{sx}, and should be considered carefully. For example, consider the glazing system supported by rigid precast concrete spandrels shown in Fig. C13.3-3. The glazing system may be subjected to full story drift, D_p, although its height ($h_x - h_y$) is only a fraction of the story height. The design drift must be accommodated by anchorage of the glazing unit, the joint between the precast spandrel and the glazing unit, or some combination of the two. Similar displacement demands arise where pipes, ducts, or conduit that are braced to the floor or roof above are connected to the top of a tall, rigid, floor-mounted component.

For ductile components, such as steel piping fabricated with welded connections, the relative seismic displacements between support points can be more significant than inertial forces. Ductile piping can accommodate relative displacements by local yielding with strain accumulations well below failure levels. However, for components fabricated using less ductile materials, where local yielding must be avoided to prevent unacceptable failure consequences, relative displacements must be accommodated by flexible connections.

C13.3.2.2 Displacements between Structures. A component or system connected to two structures must accommodate horizontal movements in any direction, as illustrated in Fig. C13.3-4.

C13.4 NONSTRUCTURAL COMPONENT ANCHORAGE

Unless exempted in Section 13.1.4, components must be anchored to the structure, and all required supports and attachments must be detailed in the construction documents. To satisfy the load path requirement of this section, the detailed information described in Section C13.2.7 must be communicated during

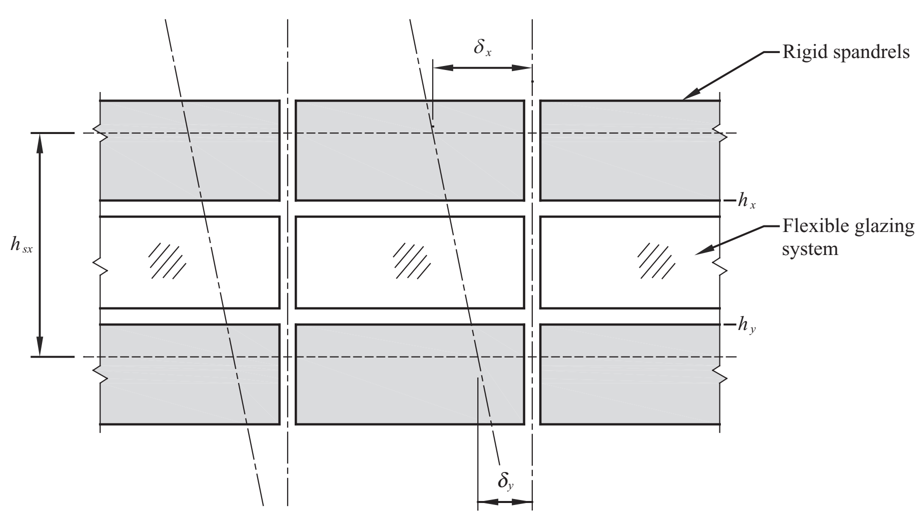

FIGURE C13.3-3 Displacements over Less than Story Height

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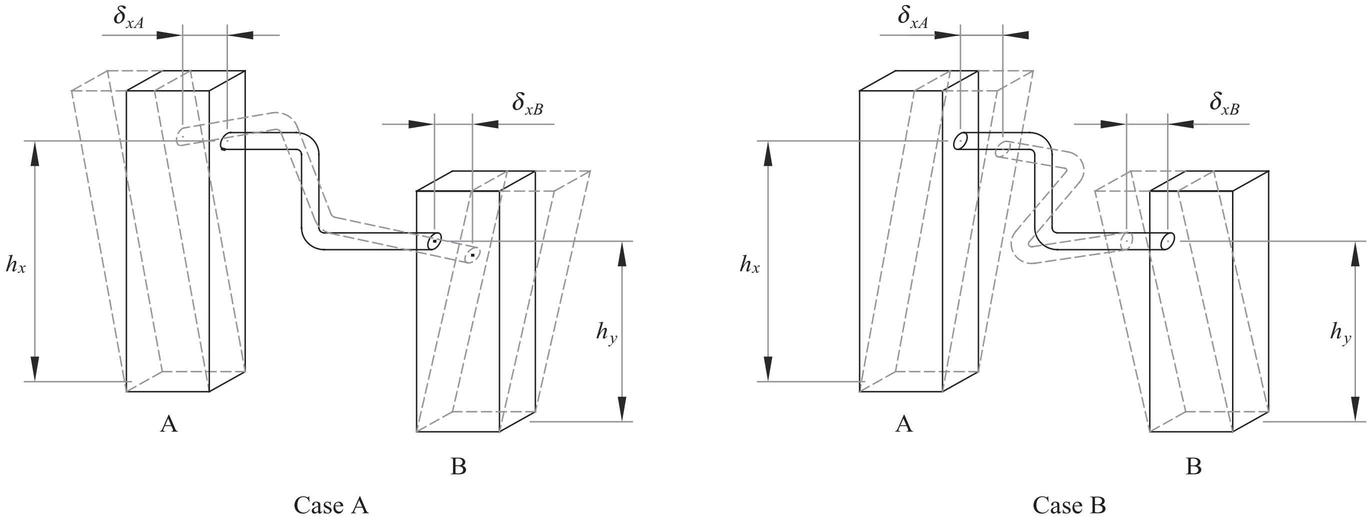

FIGURE C13.3-4 Displacements between Structures

the design phase to the registered design professional responsible for the design of the supporting structure. The load path includes housekeeping slabs and curbs, which must be adequately reinforced and positively fastened to the supporting structure. Because the exact magnitude and location of the loads imposed on the structure may not be known until nonstructural components are ordered, the initial design of supporting structural elements should be based on conservative assumptions. The design of the supporting structural elements must be verified once the final magnitude and location of the design loads have been established.

Design documents should provide details with sufficient information so that compliance with these provisions can be verified. Parameters such as a_p, R_p, I_p, S_{DS}, and W_p should be noted. Attachment details may include, as appropriate, dimensions and material properties of the connecting material, weld sizes, bolt sizes and material types for steel-to-steel connections, postinstalled anchor types, diameters, embedments, installation requirements, sheet metal screw diameters and material thicknesses of the connected parts, wood fastener types, and minimum requirements for specific gravity of the base materials.

Seismic design forces are determined using the provisions of Section 13.3.1. Specific reference standards should be consulted for additional adjustments to loads or strengths. Refer, for example, to the anchor design provisions of ACI 318, Building Code Requirements for Structural Concrete and Commentary, Appendix D for specific provisions related to seismic design of anchors in concrete. Unanchored components often rock or slide when subjected to earthquake motions. Because this behavior may have serious consequences, is difficult to predict, and is exacerbated by vertical ground motions, positive restraint must be provided for each component.

The effective seismic weight used in design of the seismic force-resisting system must include the weight of supported components. To satisfy the load path requirements of this section, localized component demand must also be considered. This satisfaction may be accomplished by checking the capacity of the first structural element in the load path (for example, a floor beam directly under a component) for combined dead, live, operating, and seismic loads, using the horizontal and vertical loads from Section 13.3.1 for the seismic demand, and repeating this procedure for each structural element or connection in the load path until the load case, including horizontal and vertical loads from Section 13.3.1, no longer governs design of the element. The load path includes housekeeping slabs and curbs, which must be adequately reinforced and positively fastened to the supporting structure.

Because the exact magnitude and location of loads imposed on the structure may not be known until nonstructural components are ordered, the initial design of supporting structural elements should be based on conservative assumptions. The design of the supporting structural elements may need to be verified once the final magnitude and location of the design loads have been established.

Tests have shown that there are consistent shear ductility variations between bolts installed in drilled or punched plates with nuts and connections using welded shear studs. The need for reductions in allowable loads for particular anchor types to account for loss of stiffness and strength may be determined through appropriate dynamic testing. Although comprehensive design recommendations are not available at present, this issue should be considered for critical connections subject to dynamic or seismic loading.

C13.4.1 Design Force in the Attachment. Previous editions of ASCE/SEI 7 included provisions for the amplification of forces to design the component anchorage. These provisions were intended to ensure that the anchorage either (a) would respond to overload in a ductile manner or (b) would be designed so that the anchorage would not be the weakest link in the load path.

Because of the difficulties associated with the application of the anchorage provisions in Section 13.4 in conjunction with anchorage provisions in other reference standards, the provisions for anchorage in ASCE/SEI 7-10 are substantially simplified. Adjustments on the R_p value used for the anchorage calculation have been eliminated, with the exception of the upper limit on R_p of 6, which is intended primarily to address the anchorage of ductile piping systems that are assigned higher R_p values. These higher component response modification factors reflect the inherent ductility and overstrength of ductile piping but may result in an underprediction of the forces on the anchorage.

C13.4.2 Anchors in Concrete or Masonry. Design capacity for anchors in concrete must be determined in accordance with ACI 318 Appendix D. Design capacity for anchors in masonry

is determined in accordance with ACI 530. Anchors must be designed to have ductile behavior or to provide a specified degree of excess strength. Depending on the specifics of the design condition, ductile design of anchors in concrete may satisfy one or more of the following objectives:

1. Adequate load redistribution between anchors in a group;
2. Allowance for anchor overload without brittle failure; or
3. Energy dissipation.

Achieving deformable, energy-absorbing behavior in the anchor itself is often difficult. Unless the design specifically addresses the conditions influencing desirable hysteretic response (e.g., adequate gauge length, anchor spacing, edge distance, and steel properties), anchors cannot be relied on for energy dissipation. Simple geometric rules, such as restrictions on the ratio of anchor embedment length to depth, may not be adequate to produce reliable ductile behavior. For example, a single anchor with sufficient embedment to force ductile tension failure in the steel body of the anchor bolt may still experience concrete fracture (a nonductile failure mode) if the edge distance is small, the anchor is placed in a group of tension-loaded anchors with reduced spacing, or the anchor is loaded in shear instead of tension. In the common case where anchors are subject primarily to shear, response governed by the steel element may be nonductile if the deformation of the anchor is constrained by rigid elements on either side of the joint. Designing the attachment so that its response is governed by a deformable link in the load path to the anchor is encouraged. This approach provides ductility and overstrength in the connection while protecting the anchor from overload. Ductile bolts should only be relied on as the primary ductile mechanism of a system if the bolts are designed to have adequate gauge length (using the unbonded strained length of the bolt) to accommodate the anticipated nonlinear displacements of the system at the design earthquake. Guidance for determining the gauge length can be found in Part 3 of the 2009 NEHRP provisions.

Anchors used to support towers, masts, and equipment are often provided with double nuts for leveling during installation. Where base-plate grout is specified at anchors with double nuts, it should not be relied on to carry loads because it can shrink and crack or be omitted altogether. The design should include the corresponding tension, compression, shear, and flexure loads.

Postinstalled anchors in concrete and masonry should be qualified for seismic loading through appropriate testing. The requisite tests for expansion and undercut anchors in concrete are given in ACI 355.2-07, *Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary*. Testing and assessment procedures based on the ACI standard that address expansion, undercut, screw, and adhesive anchors are incorporated in ICC-ES acceptance criteria. AC193, *Acceptance Criteria for Mechanical Anchors in Concrete Elements*, and AC308, *Acceptance Criteria for Post-Installed Adhesive Anchors Provisions in Concrete Elements*, refer to ACI 355.4-11, *Qualification of Post-Installed Adhesive Anchors in Concrete and Commentary*. These criteria, which include specific provisions for screw anchors and adhesive anchors, also reference ACI qualification standards for anchors. For postinstalled anchors in masonry, seismic prequalification procedures are contained in ICC-ES AC01, *Acceptance Criteria for Expansion Anchors in Masonry Elements*, AC58, *Acceptance Criteria for Adhesive Anchors in Masonry Elements*, and AC106, *Acceptance Criteria for Predrilled Fasteners (Screw Anchors) in Masonry Elements*.

Other references to adhesives (such as in Section 13.5.7.2) apply not to adhesive anchors but to steel plates and other structural elements bonded or glued to the surface of another structural component with adhesive; such connections are generally nonductile.

C13.4.3 Installation Conditions. Prying forces on anchors, which result from a lack of rotational stiffness in the connected part, can be critical for anchor design and must be considered explicitly.

For anchorage configurations that do not provide a direct mechanism to transfer compression loads (for example, a base plate that does not bear directly on a slab or deck but is supported on a threaded rod), the design for overturning must reflect the actual stiffness of base plates, equipment, housing, and other elements in the load path when computing the location of the compression centroid and the distribution of uplift loads to the anchors.

C13.4.4 Multiple Attachments. Although the standard does not prohibit the use of single anchor connections, it is good practice to use at least two anchors in any load-carrying connection whose failure might lead to collapse, partial collapse, or disruption of a critical load path.

C13.4.5 Power Actuated Fasteners. Restrictions on the use power-actuated fasteners are based on observations of failures of sprinkler pipe runs in the 1994 Northridge earthquake. Although it is unclear from the record to what degree the failures occurred because of poor installation, product deficiency, overload, or consequential damage, the capacity of power-actuated fasteners in concrete often varies more than that of drilled postinstalled anchors. The shallow embedment, small diameter, and friction mechanism of these fasteners make them particularly susceptible to the effects of concrete cracking. The suitability of power-actuated fasteners to resist tension in concrete should be demonstrated by simulated seismic testing in cracked concrete.

Where properly installed in steel, power-actuated fasteners typically exhibit reliable cyclic performance. Nevertheless, they should not be used singly to support suspended elements. Where used to attach cladding and metal decking, subassembly testing may be used to establish design capacities because the interaction among the decking, the subframe, and the fastener can only be estimated crudely by currently available analysis methods.

The exception permits the use of power-actuated fasteners for specific light-duty applications with upper limits on the load that can be resisted in these cases. All fasteners must have adequate capacity for the calculated loads, including prying forces.

The exception allows for the continued use of power-actuated fasteners in concrete for the vertical support of suspended acoustical tile or lay-in panel ceilings and for other light distributed systems such as small-diameter conduit held to the concrete surface with C-clips. Experience indicates that these applications have performed satisfactorily because of the high degree of redundancy and light loading. Other than ceilings, hung systems should not be included in this exception because of the potential for bending in the fasteners.

The exception for power-actuated fasteners in steel provides a conservative limit on loading. Currently, no accepted procedure exists for the qualification of power-actuated fasteners to resist earthquake loads.

C13.4.6 Friction Clips. The term *friction clip* is defined in Section 11.2 in a general way to encompass C-type beam clamps, as well as cold-formed metal channel (strut) connections. Friction clips are suitable to resist seismic forces provided they are properly designed and installed, but under no circumstances should they be relied on to resist sustained gravity loads. C-type clamps must be provided with restraining straps, as shown in Fig. C13.4-1.

Minimum Design Loads for Buildings and Other Structures

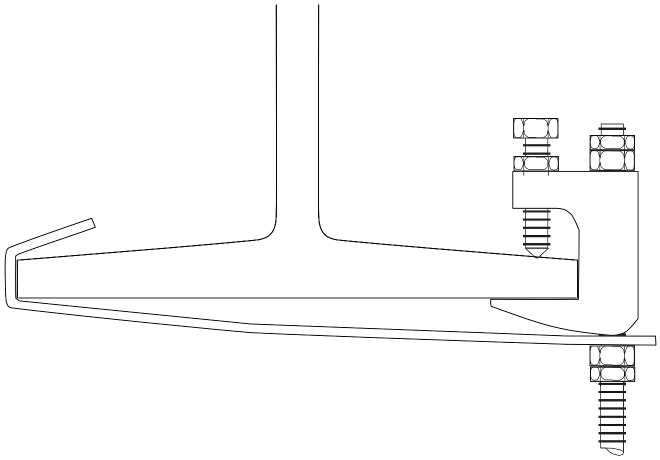

FIGURE C13.4-1 C-Type Beam Clamp Equipped with a Restraining Strap

C13.5 ARCHITECTURAL COMPONENTS

For structures in Risk Categories I through III, the requirements of Section 13.5 are intended to reduce property damage and life-safety hazards posed by architectural components and caused by loss of stability or integrity. When subjected to seismic motion, components may pose a direct falling hazard to building occupants or to people outside the building (as in the case of parapets, exterior cladding, and glazing). Failure or displacement of interior components (such as partitions and ceiling systems in exits and stairwells) may block egress.

For structures in Risk Category IV, the potential disruption of essential function caused by component failure must also be considered.

Architectural component failures in earthquakes can be caused by deficient design or construction of the component, interrelationship with another component that fails, interaction with the structure, or inadequate attachment or anchorage. For architectural components, attachment and anchorage are typically the most critical concerns related to their seismic performance. Concerns regarding loss of function are most often associated with mechanical and electrical components. Architectural damage, unless severe, can be accommodated temporarily. Severe architectural damage is often accompanied by significant structural damage.

C13.5.1 General. Suspended architectural components are not required to satisfy the force and displacement requirements of Chapter 13, where prescriptive requirements are met. The requirements were relaxed in the 2005 edition of the standard to better reflect the consequences of the expected behavior. For example, impact of a suspended architectural ornament with a sheet metal duct may only dent the duct without causing a credible danger (assuming that the ornament remains intact). The reference to Section 13.2.3 allows the designer to consider such consequences in establishing the design approach.

Nonstructural components supported by chains or otherwise suspended from the structure are exempt from lateral bracing requirements, provided that they are designed not to inflict damage to themselves or any other component when subject to seismic motion. However, for the 2005 edition, it was determined that clarifications were needed on the type of nonstructural components allowed by these exceptions and the acceptable consequences of interaction between components. In ASCE/SEI 7-02, certain nonstructural components that could represent a fire hazard after an earthquake were exempted from meeting the Section 9.6.1 requirements. For example, gas-fired space heaters clearly pose a fire hazard after an earthquake but were permitted to be exempted from the Section 9.6.1 requirements. The fire hazard after the seismic event must be given the same level of consideration as the structural failure hazard when considering components to be covered by this exception. In addition, the ASCE/SEI 7-02 language was sometimes overly restrictive because it did not distinguish between credible seismic interactions and incidental interactions. In ASCE/SEI 7-02, if a suspended lighting fixture could hit and dent a sheet metal duct, it would have to be braced, although no credible danger is created by the impact. The new reference in Section 13.2.3 of ASCE/SEI 7-05 allowed the designer to consider whether the failures of the component and/or the adjacent components are likely to occur if contact is made. These provisions have been brought into ASCE/SEI 7-10.

C13.5.2 Forces and Displacements. Partitions and interior and exterior glazing must accommodate story drift without failure that will cause a life-safety hazard. Design judgment must be used to assess potential life-safety hazards and the likelihood of life-threatening damage. Special detailing to accommodate drift for typical gypsum board or demountable partitions is unlikely to be cost-effective, and damage to these components poses a low hazard to life safety. Damage in these partitions occurs at low drift levels but is inexpensive to repair.

If they must remain intact after strong ground motion, nonstructural fire-resistant enclosures and fire-rated partitions require special detailing that provides isolation from the adjacent or enclosing structure for deformation equivalent to the calculated drift (relative displacement). In-plane differential movement between structure and wall is permitted. Provision must be made for out-of-plane restraint. These requirements are particularly important in steel or concrete moment-frame structures, which experience larger drifts. The problem is less likely to be encountered in stiff structures, such as those with shear walls.

Differential vertical movement between horizontal cantilevers in adjacent stories (such as cantilevered floor slabs) has occurred in past earthquakes. The possibility of such effects should be considered in the design of exterior walls.

C13.5.3 Exterior Nonstructural Wall Elements and Connections. Nonbearing wall panels that are attached to and enclose the structure must be designed to resist seismic (inertial) forces, wind forces, and gravity forces and to accommodate movements of the structure resulting from lateral forces and temperature change. The connections must allow wall panel movements caused by thermal and moisture changes and must be designed to prevent the loss of load-carrying capacity in the event of significant yielding. Where wind loads govern, common practice is to design connectors and panels to allow for not less than two times the story drift caused by wind loads determined, using a return period appropriate to the site location.

Design to accommodate seismic relative displacements often presents a greater challenge than design for strength. Story drifts can amount to 2 in. (50 mm) or more. Separations between adjacent panels are intended to limit contact and resulting panel misalignment or damage under all but extreme building response. Section 13.5.3, item 1 calls for a minimum separation of 1/2 in. (13 mm). For practical joint detailing and acceptable appearance, separations typically are limited to about 3/4 in. (19 mm). Manufacturing and construction tolerances for both wall elements and the supporting structure must be considered in establishing design joint dimensions and connection details.

Cladding elements, which are often very stiff in-plane, must be isolated so that they do not restrain and are not loaded by drift

of the supporting structure. Slotted connections can provide isolation, but connections with long rods that flex achieve the desired behavior without requiring precise installation. Such rods must be designed to resist tension and compression in addition to induced flexural stresses, brittle, low-cycle fatigue failure.

Full-story wall panels are usually rigidly attached to and move with the floor structure nearest the panel bottom and isolated at the upper attachments. Panels also can be vertically supported at the top connections with isolation connections at the bottom. An advantage of this configuration is that failure of an isolation connection is less likely to result in complete detachment of the panel, because it will tend to rotate into the structure rather than away from it.

To minimize the effects of thermal movements and shrinkage on architectural cladding panels, connection systems are generally detailed to be statically determinate. Because the resulting support systems often lack redundancy, exacerbating the consequences of a single connection failure, fasteners must be designed for amplified forces and connecting members must be ductile. The intent is to keep inelastic behavior in the connecting members while the more brittle fasteners remain essentially elastic. To achieve this intent, the tabulated a_p and R_p values produce fastener design forces that are about three times those for the connecting members.

Limited deformability curtain walls, such as aluminum systems, are generally light and can undergo large deformations without separating from the structure. However, care must be taken in design of these elements so that low deformability components (as defined in Section 11.2) that may be part of the system, such as glazing panels, are detailed to accommodate the expected deformations without failure.

In Table 13.5-1, veneers are classified as either limited or low deformability elements. Veneers with limited deformability, such as vinyl siding, pose little risk. Veneers with low deformability, such as brick and ceramic tile, are highly sensitive to the performance of the supporting substrate. Significant distortion of the substrate results in veneer damage, possibly including separation from the structure. The resulting risk depends on the size and weight of fragments likely to be dislodged and on the height from which the fragments would fall. Detachment of large portions of the veneer can pose a significant risk to life. Such damage can be reduced by isolating veneer from displacements of the supporting structure. For structures with flexible lateral force-resisting systems, such as moment frames and buckling-restrained braced frames, approaches used to design nonbearing wall panels to accommodate story drift should be applied to veneers.

C13.5.5 Out-of-Plane Bending. The effects of out-of-plane application of seismic forces (defined in Section 13.3.1) on nonstructural walls, including the resulting deformations, must be considered. Where weak or brittle materials are used, conventional deflection limits are expressed as a proportion of the span. The intent is to preclude out-of-plane failure of heavy materials (such as brick or block) or applied finishes (such as stone or tile).

C13.5.6 Suspended Ceilings. Suspended ceiling systems are fabricated using a wide range of building materials with differing characteristics. Some systems (such as gypsum board, screwed or nailed to suspended members) are fairly homogeneous and should be designed as light-frame diaphragm assemblies, using the forces of Section 13.3 and the applicable material-specific design provisions of Chapter 14. Others comprise discrete elements laid into a suspension system and are the subject of this section.

Seismic performance of ceiling systems with lay-in or acoustical panels depends on support of the grid and individual panels at walls and expansion joints, integrity of the grid and panel assembly, interaction with other systems (such as fire sprinklers), and support for other nonstructural components (such as light fixtures and HVAC systems). Observed performance problems include dislodgement of tiles because of impact with walls and water damage (sometimes leading to loss of occupancy) because of interaction with fire sprinklers.

Suspended lath and plaster ceilings are not exempted from the requirements of this section because of their more significant mass and the greater potential for harm associated with their failure. However, the prescriptive seismic provisions of Section 13.5.6.2 and ASTM E580 for acoustical tile and lay-in panel ceilings, including the use of compression posts, are not directly applicable to these systems primarily because of their behavior as a continuous diaphragm and greater mass. As such, they require more attention to design and detailing, in particular for the attachment of the hanger wires to the structure and main carriers, the attachment of the cross furring channels to main carriers, and the attachment of lath to cross furring channels. Attention should also be given to the attachment of light fixtures and diffusers to the ceiling structure. Bracing should consider both horizontal and vertical movement of the ceiling, as well as discontinuities and offsets. The seismic design and detailing of lath and plaster ceilings should use rational engineering methods to transfer seismic design ceiling forces to the building structural elements.

The performance of ceiling systems is affected by the placement of seismic bracing and the layout of light fixtures and other supported loads. Dynamic testing has demonstrated that splayed wires, even with vertical compression struts, may not adequately limit lateral motion of the ceiling system caused by straightening of the end loops. Construction problems include slack installation or omission of bracing wires caused by obstructions. Other testing has shown that unbraced systems may perform well where the system can accommodate the expected displacements, by providing both sufficient clearance at penetrations and wide closure members, which are now required by the standard.

With reference to the exceptions in 13.5.6,

- The first exemption is based on the presumption that lateral support is accomplished by the surrounding walls for areas equal to or less than 144 ft^2 (e.g., a 12-ft by 12-ft room). The 144 ft^2 limit corresponds historically to an assumed connection strength of 180 lb and forces associated with requirements for suspended ceilings that first appeared in the 1976 Uniform Building Code.
- The second exemption assumes that planar, horizontal drywall ceilings behave as diaphragms (i.e., develop in-plane strength). This assumption is supported by the performance of drywall ceilings in past earthquakes.

C13.5.6.1 Seismic Forces. Where the weight of the ceiling system is distributed nonuniformly, that condition should be considered in the design, because the typical T-bar ceiling grid has limited ability to redistribute lateral loads.

C13.5.6.2 Industry Standard Construction for Acoustical Tile or Lay-In Panel Ceilings. The key to good seismic performance is sufficiently wide closure angles at the perimeter to accommodate relative ceiling motion and adequate clearance at penetrating components (such as columns and piping) to avoid concentrating restraining loads on the ceiling system.

Table C13.5.6.1 provides an overview of the combined requirements of ASCE/SEI 7 and ASTM (2010). Careful

Table C13.5.6.1 Summary of Requirements for Acoustical Tile or Lay-in Panel Ceilings[1,2]

Item	Seismic Design Category C	Seismic Design Categories D, E, & F
Up to 144 sq. ft.		
NA	No requirements.	No requirements.
Greater than 144 sq. ft. but less than or equal to 1000 sq. ft.		
Duty Rating	Only Intermediate or Heavy Duty Load Rated grid as defined by ASTM C 635 may be used for commercial ceilings. (ASTM C635 sections 4.1.3.1, 4.1.3.2, & 4.1.3.3)	Heavy Duty Load Rating as defined in ASTM C 635 is required. (§5.1.1)
Grid Connections	Minimum main tee connection and cross tee intersection strength of 60 lb. (§4.1.2)	Minimum main tee connection and cross tee intersection strength of 180 lb. (§5.1.2)
Vertical Suspension Wires	Vertical hanger wires must be a minimum 12-gauge. (§4.3.1) Vertical hanger wires maximum 4 ft on center. (§4.3.1) Vertical hanger wires must be sharply bent and wrapped with three turns in three inches or less. (§4.3.2) All vertical hanger wires may not be more than 1 in 6 out of plumb without having additional wires counter-splayed. (§4.3.3) Wires may not attach to or bend around interfering equipment without the use of trapezes. (§4.3.4)	Vertical hanger wire must be a minimum 12-gauge. (§5.2.7.1) Vertical hanger wires maximum 4 ft on center. (§5.2.7.1) Vertical hanger wires must be sharply bent and wrapped with three turns in three inches or less. (§5.2.7.2) All vertical hanger wires may not be more than 1 in 6 out of plumb without having additional wires counter-splayed. (§5.2.7.3) Any connection device from the vertical hanger wire to the structure above must sustain a minimum load of 100 lb. (§5.2.7.2) Wires may not attach to or bend around interfering equipment without the use of trapezes. (§5.2.7.4)
Lateral Bracing	Lateral bracing is not permitted. Ceiling is intended to “float” relative to balance of structure. Tee connections may be insufficient to maintain integrity if braces were included. (§4.2.6, NOTE 1)	Not required under 1000 sq. ft. For ceiling areas under 1000 sq. ft., perimeter and tee connections are presumed to be sufficiently strong to maintain integrity whether bracing is installed or not. (§5.2.8)
Perimeter	Perimeter closure (molding) width must be a minimum 7/8-inch. (§4.2.2) Perimeter closures with a support ledge of less than 7/8-inch shall be supported by perimeter vertical hanger wires not more than 8 inches from the wall. (§4.2.3) A minimum clearance of 3/8-inch must be maintained on all four sides. (§4.2.4) Grid ends on all four walls must be free to move. (§4.2.6) Proprietary solutions may utilize approved attachment devices on some walls and varying closure widths. Perimeter tee ends must be tied together to prevent spreading. (§4.2.5)	Perimeter closure (molding) width must be a minimum of 2 inches. (§5.2.2) Proprietary solutions using approved perimeter clips may utilize perimeter closures less than 2 inches. (ASCE 7-10 para. 13.5.6.2.2 a) Two, adjacent sides must be connected to the wall or perimeter closure. (§5.2.3) A minimum clearance of 3/4 inch must be maintained on the other two adjacent sides. (§5.2.3) Perimeter tees must be supported by vertical hanger wires not more than 8 inches from the wall. (§5.2.6) Perimeter tee ends must be tied together not more than 8 inches from the wall to prevent spreading. (§5.2.4)
Light Fixtures	Light fixtures must be positively attached to the grid by at least two connections each capable of supporting the weight of the lighting fixture. (§4.4.1 and NEC) Surface mounted light fixtures shall be positively clamped to the grid. (§4.4.2) Clamping devices for surface mounted light fixtures shall have safety wires to the grid or the structure above. (§4.4.2) Light fixtures and attachments weighing 10 lb or less require number 12 gauge (minimum) hanger wire connected to the housing (e.g. canister light fixture). This wire may be slack. (§4.4.3) Light fixtures weighing 10 to 56 lb require two number 12 gauge (minimum) hanger wires at diagonal corners. These wires may be slack. (§4.4.4) Light fixtures greater than 56 lb require independent support from the structure. (§4.4.5) Pendent-hung light fixtures shall be supported by a minimum 9-gauge wire or other approved alternate. (§4.4.6) Rigid conduit is not permitted for the attachment of fixtures. (§4.4.7)	Light fixtures must be positively attached to the grid by at least two connections each capable of supporting the weight of the lighting fixture. (NEC, para. 5.3.1) Surface mounted light fixtures shall be positively clamped to the grid. (§5.3.2) Clamping devices for surface mounted light fixtures shall have safety wires to the grid or the structure above. (§5.3.2) When cross tees with a load carrying capacity of less than 16 lb/foot are used, supplementary hanger wires are required. (§5.3.3) Light fixtures and attachments weighing 10 lb or less require one 12 gauge minimum hanger wire connected to the housing (e.g. canister light fixture) and connected to the structure above. This wire may be slack. (§5.3.4) Light fixtures weighing 10 to 56 lb require two number 12 gauge minimum hanger wires attached to the fixture housing and connected to the structure above. These wires may be slack. (§5.3.5) Light fixtures greater than 56 lb require independent support from the structure by approved hangers. (§5.3.6) Pendent-hung light fixtures shall be supported by a minimum 9-gauge wire or other approved alternate. (§5.3.7) Rigid conduit is not permitted for the attachment of fixtures. (§5.3.8)
Mechanical Services	Flexibly mounted mechanical services weighing less than 20 lb must be positively attached to main runners or cross runners with the same load-carrying capacity as the main runners. (§4.5.1) Flexibly mounted mechanical services weighing more than 20 lb but less than 56 lb or less require two 12 gauge (minimum) hanger wires. These wires may be slack. (§4.5.2) Flexibly mounted mechanical services greater than 56 lb require direct support from the structure. (§4.5.3)	Flexibly mounted mechanical services weighing less than 20 lb must be positively attached to main runners or cross runners with the same load carrying capacity as the main runners. (§5.4.1) Flexibly mounted mechanical services weighing more than 20 lb but less than or equal to 56 lb require two 12 gauge (minimum) hanger wires. These wires may be slack. (§5.4.2) Flexibly mounted mechanical services greater than 56 lb require direct support from the structure. (§5.4.3)
Supplemental Requirements	All ceiling penetrations must have a minimum of 3/8 inch clearance on all sides. (§4.2.4)	Direct concealed systems must have stabilizer bars a maximum of 60 inches on center with stabilizer bars within 24 inches of the perimeter. (§5.2.5) Bracing is required for ceiling plane elevation changes. (§5.2.8.6) Cable trays and electrical conduits shall be supported independently of the ceiling. (§5.2.8.7) Seismic separation joints or full height partitions are required for ceiling areas greater than 2,500 sq. ft. (§5.2.9.1)

Table C13.5.6.1 (*Continued*)

Item	Seismic Design Category C	Seismic Design Categories D, E, & F
		All ceiling penetrations and independently supported fixtures or services must have closures that allow for a 1-inch movement. (§5.2.8.5) An integral ceiling sprinkler system may be designed by the licensed design professional to eliminate the required spacings of penetrations. (§5.2.8.8) A licensed design professional must review the interaction of non-essential ceiling components with essential ceiling components to prevent the failure of the essential components. (§5.7.1)
Partitions	The ceiling may not provide lateral support to partitions. (§4.6.1) Partitions attached to the ceiling must use flexible connections to avoid transferring force to the ceiling. (§4.6.1)	Partition bracing must be independent of the ceiling. (§5.5.1)
Exceptions	The ceiling weight must be less than 2.5 lb/sq. ft., otherwise the prescribed construction for Seismic Design Categories D, E, and F must be used. (§4.1.1)	None.
Greater than 1000 sq. ft. but less than or equal to 2500 sq. ft.		
Lateral Bracing	No additional requirements.	Lateral force bracing (splay wires or rigid bracing) is required within 2 inches of main tee / cross tee intersection and splayed 90 degrees apart in the plan view, at maximum 45 degree angle from the horizontal and located 12 ft. on center in both directions, starting 6 ft. from walls. (§5.2.8.1 & §5.2.8.2) Lateral force bracing must be spaced a minimum of 6 inches from unbraced horizontal piping or ductwork. (§5.2.8.3) Lateral force bracing connection strength must be a minimum of 250 lb. (§5.2.8.3) Unless rigid bracing is used or calculations have shown that lateral deflection is less than 1/4 inch, sprinkler heads and other penetrations shall have a minimum of 1 inch clear space in all directions. (§5.2.8.5)
Greater than 2500 sq. ft.		
Special Considerations	No additional requirements.	Seismic separation joints with a minimum or 3/4-inch axial movement or full height partitions with the usual 2-inch closure and other requirements. (§5.2.9.1)

[1]There are no requirements for suspended ceilings located in structures assigned to Seismic Design Categories A and B.
[2]Unless otherwise noted, all section references in parentheses (§) refer to ASTM E580 (2006).

review of both documents is required to determine the actual requirements.

C13.5.6.2.1 Seismic Design Category C. The prescribed method for SDC C is a floating ceiling. The design assumes a small displacement of the building structure caused by the earthquake at the ceiling and isolates the ceiling from the perimeter. The vertical hanger wires are not capable of transmitting significant movement or horizontal force into the ceiling system and therefore the ceiling does not experience significant force or displacement as long as the perimeter gap is not exceeded. All penetrations and services must be isolated from the building structure for this construction method to be effective. If this isolation is impractical or undesirable, the prescribed construction for SDCs D, E, and F may be used.

C13.5.6.2.2 Seismic Design Categories D through F. The industry standard construction addressed in this section relies on ceiling contact with the perimeter wall for restraint.

Typical splay wire lateral bracing allows for some movement before it effectively restrains the ceiling. The intent of the 2-in. perimeter closure wall angle is to permit back-and-forth motion of the ceiling during an earthquake without loss of support, and the width of the closure angle is key to good performance. This standard has been experimentally verified by large-scale testing conducted by ANCO Engineering in 1983.

Extensive shake-table testing using the protocol contained in ICC-ES AC156 by major manufacturers of suspended ceilings has shown that perimeter clips can provide equivalent performance if they are designed to accommodate the same degree of movement as the closure angle while supporting the T ends.

The requirement for a 1-in. clearance around sprinkler drops found in 13.5.6.2.2 (e) of ASCE/SEI 7-05 is maintained and is contained in ASTM E580.

This seismic separation joint is intended to break the ceiling into isolated areas, preventing large-scale force transfer across the ceiling. The new requirement to accommodate 3/4-in. axial movement specifies the performance requirement for the separation joint.

The requirement for seismic separation joints to limit ceiling areas to 2,500 ft^2 is intended to prevent overload of the connections to the perimeter angle. Limiting the ratio of long to short dimensions to 4:1 prevents dividing the ceiling into long and narrow sections, which could defeat the purpose of the separation.

C13.5.6.3 Integral Construction. Ceiling systems that use integral construction are constructed of modular preengineered components that integrate lights, ventilation components, fire sprinklers, and seismic bracing into a complete system. They may include aluminum, steel, and PVC components and may be

designed using integral construction of ceiling and wall. They often use rigid grid and bracing systems, which provide lateral support for all the ceiling components, including sprinkler drops. This bracing reduces the potential for adverse interactions among components and eliminates the need to provide clearances for differential movement.

C13.5.7 Access Floors

C13.5.7.1 General. In past earthquakes and in cyclic load tests, some typical raised access floor systems behaved in a brittle manner and exhibited little reserve capacity beyond initial yielding or failure of critical connections. Testing shows that unrestrained individual floor panels may pop out of the supporting grid unless they are mechanically fastened to supporting pedestals or stringers. This fault may be a concern, particularly in egress pathways.

For systems with floor stringers, it is accepted practice to calculate the seismic force, F_p, for the entire access floor system within a partitioned space and then distribute the total force to the individual braces or pedestals. For stringerless systems, the seismic load path should be established explicitly.

Overturning effects subject individual pedestals to vertical loads well in excess of the weight, W_p, used in determining the seismic force, F_p. It is unconservative to use the design vertical load simultaneously with the design seismic force for design of anchor bolts, pedestal bending, and pedestal welds to base plates. "Slip on" heads that are not mechanically fastened to the pedestal shaft and thus cannot transfer tension are likely unable to transfer to the pedestal the overturning moments generated by equipment attached to adjacent floor panels.

To preclude brittle failure, each element in the seismic load path must have energy-absorbing capacity. Buckling failure modes should be prevented. Lower seismic force demands are allowed for special access floors that are designed to preclude brittle and buckling failure modes.

C13.5.7.2 Special Access Floors. An access floor can be a "special access floor" if the registered design professional opts to comply with the requirements of Section 13.5.7.2. Special access floors include construction features that improve the performance and reliability of the floor system under seismic loading. The provisions focus on providing an engineered load path for seismic shear and overturning forces. Special access floors are designed for smaller lateral forces, and their use is encouraged at facilities with higher nonstructural performance objectives.

C13.5.8 Partitions. Partitions subject to these requirements must have independent lateral support bracing from the top of the partition to the building structure or to a substructure attached to the building structure. Some partitions are designed to span vertically from the floor to a suspended ceiling system. The ceiling system must be designed to provide lateral support for the top of the partition. An exception to this condition is provided to exempt bracing of light (gypsum board) partitions where the load does not exceed the minimum partition lateral load. Experience has shown that partitions subjected to the minimum load can be braced to the ceiling without failure.

C13.5.9 Glass in Glazed Curtain Walls, Glazed Storefronts, and Glazed Partitions. The performance of glass in earthquakes falls into one of four categories:

1. The glass remains unbroken in its frame or anchorage.
2. The glass cracks but remains in its frame or anchorage while continuing to provide a weather barrier and to be otherwise serviceable.
3. The glass shatters but remains in its frame or anchorage in a precarious condition, likely to fall out at any time.
4. The glass falls out of its frame or anchorage, either in shards or as whole panels.

Categories 1 and 2 satisfy both immediate-occupancy and life-safety performance objectives. Although the glass is cracked in Category 2, immediate replacement is not required. Categories 3 and 4 cannot provide for immediate occupancy, and their provision of life safety depends on the postbreakage characteristics of the glass and the height from which it can fall. Tempered glass shatters into multiple, pebble-size fragments that fall from the frame or anchorage in clusters. These broken glass clusters are relatively harmless to humans when they fall from limited heights, but they could be harmful when they fall from greater heights.

C13.5.9.1 General. Equation 13.5-1 is derived from Sheet Glass Association of Japan (1982) and is similar to an equation in Bouwkamp and Meehan (1960) that permits calculation of the story drift required to cause glass-to-frame contact in a given rectangular window frame. Both calculations are based on the principle that a rectangular window frame (specifically, one that is anchored mechanically to adjacent stories of a structure) becomes a parallelogram as a result of story drift, and that glass-to-frame contact occurs when the length of the shorter diagonal of the parallelogram is equal to the diagonal of the glass panel itself. The value $\Delta_{fallout}$ represents the displacement capacity of the system, and D_p represents the displacement demand.

The 1.25 factor in the requirements described above reflect uncertainties associated with calculated inelastic seismic displacements of building structures. Wright (1989) states that

> post-elastic deformations, calculated using the structural analysis process may well underestimate the actual building deformation by up to 30 percent. It would therefore be reasonable to require the curtain wall glazing system to withstand 1.25 times the computed maximum interstory displacement to verify adequate performance.

The reason for the second exception to Eq. 13.5-1 is that the tempered glass, if shattered, would not produce an overhead falling hazard to adjacent pedestrians, although some pieces of glass may fall out of the frame.

C13.5.9.2 Seismic Drift Limits for Glass Components. As an alternative to the prescriptive approach of Section 13.5.9.1, the deformation capacity of glazed curtain wall systems may be established by test.

C13.6 MECHANICAL AND ELECTRICAL COMPONENTS

These requirements, focused on design of supports and attachments, are intended to reduce the hazard to life posed by loss of component structural stability or integrity. The requirements increase the reliability of component operation but do not address functionality directly. For critical components where operability is vital, Section 13.2.2 provides methods for seismically qualifying the component.

Traditionally, mechanical components (such as tanks and heat exchangers) without rotating or reciprocating components are directly anchored to the structure. Mechanical and electrical equipment components with rotating or reciprocating elements are often isolated from the structure by vibration isolators (using rubber acting in shear, springs, or air cushions). Heavy mechanical equipment (such as large boilers) may not be restrained at

all, and electrical equipment other than generators, which are normally isolated to dampen vibrations, usually is rigidly anchored (for example, switch gear and motor control centers).

Two distinct levels of earthquake safety are considered in the design of mechanical and electrical components. At the usual safety level, failure of the mechanical or electrical component itself because of seismic effects poses no significant hazard. In this case, design of the supports and attachments to the structure is required to avoid a life-safety hazard. At the higher safety level, the component must continue to function acceptably after the design earthquake. Such components are defined as designated seismic systems in Section 11.2 and may be required to meet the special certification requirements of Section 13.2.2.

Not all equipment or parts of equipment need to be designed for seismic forces. Where I_p is specified to be 1.0, damage to, or even failure of, a piece or part of a component does not violate these requirements as long as a life-safety hazard is not created. The restraint or containment of a falling, breaking, or toppling component (or its parts) by means of bumpers, braces, guys, wedges, shims, tethers, or gapped restraints to satisfy these requirements often is acceptable, although the component itself may suffer damage.

Judgment is required to fulfill the intent of these requirements; the key consideration is the threat to life safety. For example, a nonessential air handler package unit that is less than 4 ft (1.2 m) tall bolted to a mechanical room floor is not a threat to life as long as it is prevented from significant displacement by having adequate anchorage. In this case, seismic design of the air handler itself is unnecessary. However, a 10-ft (3.0 m) tall tank on 6-ft (1.8 m) long angles used as legs, mounted on a roof near a building exit does pose a hazard. The intent of these requirements is that the supports and attachments (tank legs, connections between the roof and the legs, and connections between the legs and the tank), and possibly even the tank itself be designed to resist seismic forces. Alternatively, restraint of the tank by guys or bracing could be acceptable.

It is not the intent of the standard to require the seismic design of shafts, buckets, cranks, pistons, plungers, impellers, rotors, stators, bearings, switches, gears, nonpressure retaining casings and castings, or similar items. Where the potential for a hazard to life exists, the design effort should focus on equipment supports, including base plates, anchorages, support lugs, legs, feet, saddles, skirts, hangers, braces, or ties.

Many mechanical and electrical components consist of complex assemblies of parts that are manufactured in an industrial process that produces similar or identical items. Such equipment may include manufacturer's catalog items and often are designed by empirical (trial-and-error) means for functional and transportation loadings. A characteristic of such equipment is that it may be inherently rugged. The term "rugged" refers to an ampleness of construction that provides such equipment with the ability to survive strong motions without significant loss of function. By examining such equipment, an experienced design professional usually should be able to confirm such ruggedness. The results of an assessment of equipment ruggedness may be used in determining an appropriate method and extent of seismic design or qualification effort.

The revisions to Table 13-3 in ASCE/SEI 07-10 are the result of work done in recent years to better understand the performance of mechanical and electrical components and their attachment to the structure. The primary concepts of flexible and rigid equipment and ductile and rugged behavior are drawn from SEAOC (1999), Commentary Section C107.1.7. Material on HVAC is based on ASHRAE (2000). Other material on industrial piping, boilers, and pressure vessels is based on the American Society of Mechanical Engineers codes and standards publications.

C13.6.1 General. The exception allowing unbraced suspended components has been clarified, addressing concerns about the type of nonstructural components allowed by these exceptions, as well as the acceptable consequences of interaction between components. In previous editions of the standard, certain nonstructural components that could represent a fire hazard after an earthquake were exempt from lateral bracing requirements. In the revised exception, reference to Section 13.2.3 addresses such concerns while distinguishing between credible seismic interactions and incidental interactions.

The seismic demand requirements are based on component structural attributes of flexibility (or rigidity) and ruggedness. Table 13.6-1 provides seismic coefficients based on judgments of the component flexibility, expressed in the a_p term, and ruggedness, expressed in the R_p term. It may also be necessary to consider the flexibility and ductility of the attachment system that provides seismic restraint.

Entries for components and systems in Table 13.6-1 are grouped and described to improve clarity of application. Components are divided into three broad groups, within which they are further classified depending on the type of construction or expected seismic behavior. For example, mechanical components include "air-side" components (such as fans and air handlers) that experience dynamic amplification but are light and deformable; "wet-side" components that generally contain liquids (such as boilers and chillers) that are more rigid and somewhat ductile; and rugged components (such as engines, turbines, and pumps) that are of massive construction because of demanding operating loads, and generally perform well in earthquakes, if adequately anchored.

A distinction is made between components isolated using neoprene and those that are spring isolated. Spring-isolated components are assigned a lower R_p value because they tend to have less effective damping. Internally isolated components are classified explicitly to avoid confusion.

C13.6.2 Component Period. Component period is used to clarify components as rigid ($T \le 0.06$ s) or flexible ($T > 0.06$ s). Determination of the fundamental period of a mechanical or electrical component using analytical or test methods can become involved. If not properly performed, the fundamental period may be underestimated, producing unconservative results. The flexibility of the component's supports and attachments typically dominates response and thus fundamental component period. Therefore, analytical determinations of component period must consider those sources of flexibility. Where determined by testing, the dominant mode of vibration of concern for seismic evaluation must be excited and captured by the test setup. This dominant mode of vibration cannot be discovered through in situ tests that measure only ambient vibrations. To excite the mode of vibration with the highest fundamental period by in situ tests, relatively significant input levels of motion are required (that is, the flexibility of the base and attachment must be exercised). A resonant frequency search procedure, such as that given in ICC-ES AC156, may be used to identify the dominant modes of vibration of a component.

Many types of mechanical components have fundamental periods below 0.06 s and may be considered rigid. Examples include horizontal pumps, engine generators, motor generators, air compressors, and motor-driven centrifugal blowers. Other types of mechanical equipment are stiff but may have fundamental periods up to about 0.125 s. Examples include vertical immersion and deep well pumps, belt-driven and vane axial fans,

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heaters, air handlers, chillers, boilers, heat exchangers, filters, and evaporators. These fundamental period estimates do not apply where the equipment is mounted on vibration isolators.

Electrical equipment cabinets can have fundamental periods of about 0.06 to 0.3 s, depending on the supported weight and its distribution, the stiffness of the enclosure assembly, the flexibility of the enclosure base, and the load path through to the attachment points. Tall, narrow motor control centers and switchboards lie at the upper end of this period range. Low- and medium-voltage switch gear, transformers, battery chargers, inverters, instrumentation cabinets, and instrumentation racks usually have fundamental periods ranging from 0.1 to 0.2 s. Braced battery racks, stiffened vertical control panels, bench boards, electrical cabinets with top bracing, and wall-mounted panel boards have fundamental periods ranging from 0.06 to 0.1 s.

C13.6.3 Mechanical Components and C13.6.4 Electrical Components. Most mechanical and electrical equipment is inherently rugged and, where properly attached to the structure, has performed well in past earthquakes. Because the operational and transportation loads for which the equipment is designed typically are larger than those caused by earthquakes, these requirements focus primarily on equipment anchorage and attachments. However, designated seismic systems, which are required to function after an earthquake or which must maintain containment of flammable or hazardous materials, must themselves be designed for seismic forces or be qualified for seismic loading in accordance with Section 13.2.2.

The likelihood of postearthquake operability can be increased where the following measures are taken:

1. Internal assemblies, subassemblies, and electrical contacts are attached sufficiently to prevent their being subjected to differential movement or impact with other internal assemblies or the equipment enclosure.
2. Operators, motors, generators, and other such components that are functionally attached to mechanical equipment by means of an operating shaft or mechanism are structurally connected or commonly supported with sufficient rigidity such that binding of the operating shaft is avoided.
3. Any ceramic or other nonductile components in the seismic load path are specifically evaluated.
4. Adjacent electrical cabinets are bolted together and cabinet lineups are prevented from impacting adjacent structural members.

Components that could be damaged, or could damage other components, and are fastened to multiple locations of a structure, must be designed to accommodate seismic relative displacements. Such components include bus ducts, cable trays, conduits, elevator guide rails, and piping systems. As discussed in Section C13.3.2.1, special design consideration is required where full story drift demands are concentrated in a fraction of the story height.

C13.6.5 Component Supports. The intent of this section is to require seismic design of all mechanical and electrical component supports to prevent sliding, falling, toppling, or other movement that could imperil life. Component supports are differentiated here from component attachments to emphasize that the supports themselves, as enumerated in the text, require seismic design even if they are fabricated by the mechanical or electrical component manufacturer. This need exists regardless of whether the mechanical or electrical component itself is designed for seismic loads.

C13.6.5.1 Design Basis. Standard supports are those developed in accordance with a reference document (Section 13.1.6). Where standard supports are not used, the seismic design forces and displacement demands of Chapter 13 are used with applicable material-specific design procedures of Chapter 14.

C13.6.5.2 Design for Relative Displacement. For some items, such as piping, seismic relative displacements between support points are of more significance than inertial forces. Components made of high deformability materials such as steel or copper can accommodate relative displacements inelastically, provided that the connections also provide high deformability. Threaded and soldered connections exhibit poor ductility under inelastic displacements, even for ductile materials. Components made of less ductile materials can accommodate relative displacement effects only if appropriate flexibility or flexible connections are provided.

Detailing distribution systems that connect separate structures with bends and elbows makes them less prone to damage and less likely to fracture and fall, provided that the supports can accommodate the imposed loads.

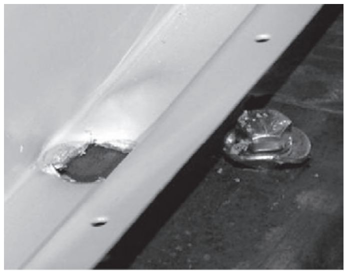
C13.6.5.3 Support Attachment to Component. As used in this section, "integral" relates to the manufacturing process, not the location of installation. For example, both the legs of a cooling tower and the attachment of the legs to the body of the cooling tower must be designed, even if the legs are provided by the manufacturer and installed at the plant. Also, if the cooling tower has an $I_p = 1.5$, the design must address not only the attachments (e.g., welds and bolts) of the legs to the component but also local stresses imposed on the body of the cooling tower by the support attachments.

C13.6.5.5 Additional Requirements. As reflected in this section of the standard and in the footnote to Table 13.6-1, vibration-isolated equipment with snubbers is subject to amplified loads as a result of dynamic impact.

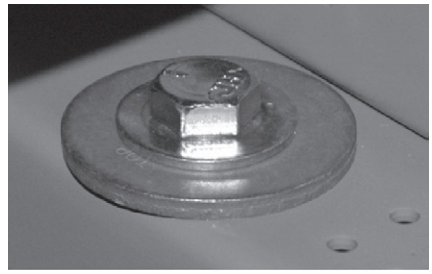
Most sheet metal connection points for seismic anchorage do not exhibit the same mechanical properties as bolted connections with structural elements. The use of Belleville washers improves the seismic performance of connections to equipment enclosures fabricated from sheet metal 7 gauge (0.18 in.) or thinner by distributing the stress over a larger surface area of the sheet metal connection interface, allowing for bolted connections to be torqued to recommended values for proper preload while reducing the tendency for local sheet metal tearing or bending failures or loosening of the bolted connection (Fig. C13.6-1). The intrinsic spring loading capacity of the Belleville washer assists with long-term preload retention to maintain integrity of the seismic anchorage.

Manufacturers test or design their equipment to handle seismic loads at the equipment "hard points" or anchor locations. The results of this design qualification effort are typically reflected in installation instructions provided by the manufacturer. It is imperative that the manufacturer's installation instructions be followed. Where such guidance does not exist, the registered design professional should design appropriate reinforcement.

C13.6.5.6 Conduit, Cable Tray, and Other Electrical Distribution Systems (Raceways). The term *raceway* is defined in several standards with somewhat varying language. As used here, it is intended to describe all electrical distribution systems including conduit, cable trays, and open and closed raceways. Experience indicates that a size limit of 2.5 in. can be established for the provision of flexible connections to accommodate seismic relative displacements that might occur between pieces of connected equipment because smaller conduit normally possesses



Failure of sheet metal base anchored with standard washer (With permission)



Anchorage equipped with Belleville washer (With permission)

FIGURE C13.6-1 Equipment Anchorage with Belleville Washers

the required flexibility to accommodate such displacements. The bracing exemption for hangers less than 12 in. (305 mm) long, presuming the hangers have negligible bending strength and sufficient resistance to lateral seismic loads, is provided by the restoring force induced by pendulum displacement of the raceway. The short length of the hangers is also presumed to limit the amount of horizontal raceway displacement.

Short hangers fabricated from threaded rods resist lateral force primarily through bending and are prone to failure through cyclic fatigue. Providing a swivel at the connection between the threaded rod hanger and the structure eliminates the bending stress in the threaded rod. Where this swivel and braces are not provided, the rod hangers (and, where applicable, the anchors) must be designed for the resultant bending forces.

The exemption for short hangers is limited to the case where every hanger in the raceway run is less than 12 in. (305 mm) because of the need to carefully consider the seismic loads and compatible displacement limits for the portions of raceways with longer hanger supports.

C13.6.6 Utility and Service Lines. For essential facilities (Risk Category IV), auxiliary on-site mechanical and electrical utility sources are recommended.

Where utility lines pass through the interface of adjacent, independent structures, they must be detailed to accommodate differential displacement computed in accordance with Section 13.3.2 and including the C_d factor of Section 12.2.1.

As specified in Section 13.1.3, nonessential piping whose failure could damage essential utilities in the event of pipe rupture may be considered designated seismic systems.

C13.6.7 Ductwork. Experience in past earthquakes has shown that HVAC duct systems are rugged and perform well in strong ground shaking. Bracing in accordance with ANSI/SMACNA 001 has been effective in limiting damage to duct systems. Typical failures have affected only system function, and major damage or collapse has been uncommon. Therefore, industry standard practices should prove adequate for most installations. Expected earthquake damage is limited to opening of duct joints and tears in ducts. Connection details that are prone to brittle failures, especially hanger rods subject to large amplitude cycles of bending stress, should be avoided.

The amplification factor for ductwork has been increased from 1.0 to 2.5 because even braced duct systems are relatively flexible. The R_p values also have been increased so that the resulting seismic design forces are consistent with those determined previously.

Ductwork systems that carry hazardous materials or must remain operational during and after an earthquake are assigned a value of $I_p = 1.5$, and they require a detailed engineering analysis addressing leak-tightness.

Lighter in-line components may be designed to resist the forces from Section 13.3 as part of the overall duct system design, whereby the duct attached to the in-line component is explicitly designed for the forces generated by the component. Where in-line components are more massive, the component must be supported and braced independently of the ductwork to avoid failure of the connections.

The requirements for flexible connections of unbraced piping to in-line components such as reheat coils applies regardless of the component weight.

C13.6.8 Piping Systems. Because of the typical redundancy of piping system supports, documented cases of total collapse of piping systems in earthquakes are rare; however, pipe leakage resulting from excessive displacement or overstress often results in significant consequential damage and in some cases loss of facility operability. Loss of fluid containment (leakage) normally occurs at discontinuities such as threads, grooves, bolted connectors, geometric discontinuities, or locations where incipient cracks exist, such as at the toe or root of a weld or braze. Numerous building and industrial national standards and guidelines address a wide variety of piping systems materials and applications. Construction in accordance with the national standards referenced in these provisions is usually effective in limiting damage to piping systems and avoiding loss of fluid containment under earthquake conditions.

ASHRAE (2000) and MSS (2001) are derived in large part from the predecessors of SMACNA (2008). These documents may be appropriate references for use in the seismic design of piping systems. Because the SMACNA standard does not refer to pipe stresses in the determination of hanger and brace spacing, however, a supplementary check of pipe stresses may be necessary when this document is used. ASME piping rules as given in the ASME (2010) and ASME B31 parts B31.1, B31.3, B31.5, B31.9, and B31.12 are normally used for high-pressure, high-temperature piping but can also conservatively be applied to other lower pressure, lower temperature piping systems. Code-compliant seismic design manuals prepared specifically for proprietary systems may also be appropriate references.

Table 13-3 entries for piping previously listed the amplification factor related to the response of piping systems as rigid (a_p = 1.0) and values for component response modification factors lower than in the current table. However, it was realized that most piping systems are flexible and that the amplification factor values should reflect this fact; thus, a_p was increased to 2.5 and the R_p values were adjusted accordingly such that a_p/R_p remains roughly consistent with earlier provisions.

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Although seismic design in accordance with Section 13.6.8 generally ensures that effective seismic forces do not fail piping, seismic displacements may be underestimated such that impact with nearby structural, mechanical, or electrical components could occur. In marginal cases, it may be advisable to protect the pipe with wrapper plates where impacts could occur, including at gapped supports. Insulation may in some cases also serve to protect the pipe from impact damage. Piping systems are typically designed for pressure containment, and piping designed with a factor of safety of three or more against pressure failure (rupture) may be inherently robust enough to survive impact with nearby structures, equipment, and other piping, particularly if the piping is insulated. Piping that has less than standard weight wall thickness may require the evaluation of the effects of impact locally on the pipe wall and may necessitate means to protect the pipe wall.

It is usually preferable for piping to be detailed to accommodate seismic relative displacements between the first seismic support upstream or downstream from connections and other seismically supported components or headers. This accommodation is preferably achieved by means of pipe flexibility or, where pipe flexibility is not possible, flexible supports. Piping not otherwise detailed to accommodate such seismic relative displacements must be provided with connections that have sufficient flexibility in the connecting element or in the component or header to avoid failure of the piping. The option to use a flexible connecting element may be less desirable because of the need for greater maintenance efforts to ensure continued proper function of the flexible element.

Grooved couplings, ball joints, resilient gasket compression fittings, other articulating-type connections, bellows expansion joints, and flexible metal hose are used in many piping systems and can serve to increase the rotational and lateral deflection design capacity of the piping connections.

Grooved couplings are classified as either rigid or flexible. Flexible grooved couplings demonstrate limited free rotational capacity. The free rotational capacity is the maximum articulating angle where the connection behaves essentially as a pinned joint with limited or negligible stiffness. The remaining rotational capacity of the connection is associated with conventional joint behavior, and design force demands in the connection are determined by traditional means.

Rigid couplings are typically used for high-pressure applications and usually are assumed to be stiffer than the pipe. Alternatively, rigid coupling may exhibit bilinear rotational stiffness with the initial rotational stiffness affected by installation.

Coupling flexibilities vary significantly between manufacturers, particularly for rigid couplings. Manufacturer's data may be available. Industrywide procedures for the determination of coupling flexibility are not currently available; however, some guidance for couplings may be found in the provisions for fire sprinkler piping, where grooved couplings are classified as either rigid or flexible on the basis of specific requirements on angular movement. In Section 3.5.4 of NFPA (2007), flexible couplings are defined as follows:

"A listed coupling or fitting that allows axial displacement, rotation, and at least 1 degree of angular movement of the pipe without inducing harm on the pipe. For pipe diameters of 8 in. (203.2 mm) and larger, the angular movement shall be permitted to be less than 1 degree but not less than 0.5 degrees."

Couplings determined to be flexible on this basis are listed either with FM Global (2007) or UL (2004).

Piping component testing suggests that the ductility capacity of carbon steel threaded and flexible grooved piping component joints ranges between 1.4 and 3.0, implying an effective stress intensification of approximately 2.5. These types of connections have been classified as having limited deformability, and piping systems with these connections have R_p values lower than piping with welded or brazed joints.

The allowable stresses for piping constructed with ductile materials assumed to be materials with high deformability, and not designed in accordance with an applicable standard or recognized design basis, are based on values consistent with industrial piping and structural steel standards for comparable piping materials.

The allowable stresses for piping constructed with low-deformability materials, and not designed in accordance with an applicable standard or recognized design basis, are derived from values consistent with ASME standards for comparable piping materials.

For typical piping materials, pipe stresses may not be the governing parameter in determining the hanger and other support spacing. Other considerations, such as the capacity of the hanger and other support connections to the structure, limits on the lateral displacements between hangers and other supports to avoid impacts, the need to limit pipe sag between hangers to avoid the pooling of condensing gases, and the loads on connected equipment may govern the design. Nevertheless, seismic span tables, based on limiting stresses and displacements in the pipe, can be a useful adjunct for establishing seismic support locations.

Piping systems' service loads of pressure and temperature also need to be considered in conjunction with seismic inertia loads. The potential for low ambient and lower than ambient operating temperatures should be considered in the designation of the piping system materials as having high or low deformability. High deformability may often be assumed for steels, particularly ASME listed materials operating at high temperatures, copper and copper alloys, and aluminum. Low deformability should be assumed for any piping material that exhibits brittle behavior, such as glass, ceramics, and many plastics.

Piping should be designed to accommodate relative displacements between the first rigid piping support and connections to equipment or piping headers often assumed to be anchors. Barring such design, the equipment or header connection could be designed to have sufficient flexibility to avoid failure. The specification of such flexible connections should consider the necessity of connection maintenance.

Where appropriate, a walkdown of the finally installed piping system by an experienced design professional familiar with seismic design is recommended, particularly for piping greater than 6 in. (152.4 mm) nominal pipe size, high-pressure piping, piping operating at higher than ambient temperatures, and piping containing hazardous materials. The need for a walkdown may also be related to the scope, function, and complexity of the piping system, as well as the expected performance of the facility. In addition to providing a review of seismic restraint location, orientation, and attachment to the structure, the walkdown verifies that the required separation exists between the piping and nearby structures, equipment, and other piping in the as-built condition.

C13.6.8.1 ASME Pressure Piping Systems. In Table 13-3, the increased R_p values listed for ASME B31-compliant piping systems are intended to reflect the more rigorous design, construction, and quality control requirements, as well as the intensified stresses associated with ASME B31 designs.

Materials meeting ASME toughness requirements may be considered high-deformability materials.

C13.6.8.2 Fire Protection Sprinkler Piping Systems. The lateral design procedures of NFPA (2007) have been revised for consistency with the ASCE/SEI 7 design approach while retaining traditional sprinkler system design concepts. Using conservative upper-bound values of the various design parameters, a single lateral force coefficient, C_p, was developed. It is a function of the mapped short period response parameter S_s. Stresses in the pipe and connections are controlled by limiting the maximum reaction at bracing points as a function of pipe diameter.

Other components of fire protection systems, e.g., pumps and control panels, are subject to the general requirements of ASCE/SEI 7.

C13.6.8.3 Exceptions. The conditions under which the force and displacement requirements of Section 13.3 may be waived are based on observed performance in past earthquakes. The 12-in. (305-mm) limit on the hanger or trapeze drop must be met by all the hangers or trapezes supporting the piping system.

C13.6.9 Boilers and Pressure Vessels. Experience in past earthquakes has shown that boilers and pressure vessels are rugged and perform well in strong ground motion. Construction in accordance with current requirements of the ASME *Boiler and Pressure Vessel Code* (ASME BPVC) has been shown to be effective in limiting damage to and avoiding loss of fluid containment in boilers and pressure vessels under earthquake conditions. It is, therefore, the intent of the standard that nationally recognized codes be used to design boilers and pressure vessels provided that the seismic force and displacement demands are equal to or exceed those outlined in Section 13.3. Where nationally recognized codes do not yet incorporate force and displacement requirements comparable to the requirements of Section 13.3, it is nonetheless the intent to use the design acceptance criteria and construction practices of those codes.

C13.6.10 Elevator and Escalator Design Requirements. The ASME *Safety Code for Elevators and Escalators* (ASME A17.1) has adopted many requirements to improve the seismic response of elevators; however, they do not apply to some regions covered by this chapter. These changes are to extend force requirements for elevators to be consistent with the standard.

C13.6.10.3 Seismic Controls for Elevators. ASME A17.1 Section 8.4.10.1.2 specifies the requirements for the location and sensitivity of seismic switches to achieve the following goals: (a) safe shutdown in the event of an earthquake severe enough to impair elevator operations, (b) rapid and safe reactivation of elevators after an earthquake, and (c) avoidance of unintended elevator shutdowns. This level of safety is achieved by requiring the switches to be in or near the elevator equipment room, by using switches located on or near building columns that respond to vertical accelerations that would result from *P* and *S* waves, and by setting the sensitivity of the switches at a level that avoids false shutdowns because of nonseismic sources of vibration. The trigger levels for switches with horizontal sensitivity (for cases where the switch cannot be located near a column) are based on the experience with California hospitals in the Northridge earthquake of 1994. Elevators in which the seismic switch and counterweight derail device have triggered should not be put back into service without a complete inspection. However, in the case where the loss of use of the elevator creates a life-safety hazard, an attempt to put the elevator back into service may be attempted. Operating the elevator before inspection may cause severe damage to the elevator or its components.

The building owner should have detailed written procedures in place defining for the elevator operator and/or maintenance personnel which elevators in the facility are necessary from a postearthquake life-safety perspective. It is highly recommended that these procedures be in place, with appropriate personnel training, before an event strong enough to trip the seismic switch.

C13.6.10.4 Retainer Plates. The use of retainer plates is a low-cost provision to improve the seismic response of elevators.

C13.6.11 Other Mechanical and Electrical Components. The material properties set forth in item 2 of this section are similar to those allowed in ASME BPVC and reflect the high factors of safety necessary for seismic, service, and environmental loads.

REFERENCES

American Concrete Institute (ACI). (2007). "Qualification of post-installed mechanical anchors in concrete and commentary." *ACI 355.2*, Farmington Hills, Mich.

American Concrete Institute (ACI). (2011a). "Building code requirements and specification for masonry structures and related commentaries." *ACI 530/530.1*, Farmington Hills, Mich.

American Concrete Institute (ACI). (2011b). "Building code requirements for structural concrete and commentary." *ACI 318*, Farmington Hills, Mich.

American Concrete Institute (ACI). (2011c). "Qualification of post-installed adhesive anchors in concrete and commentary." *ACI 355.4*, Farmington Hills, Mich.

American National Standards Institute/Sheet Metal and Air-Conditioning Contractors National Association (ANSI/SMACNA). (2000). "Seismic restraint manual: Guidelines for mechanical systems." *ANSI/SMACNA 001*, Chantilly, Va.

American Society of Heating, Refrigerating, and Air-Conditioning Engineers (ASHRAE). (2000). *Practical Guide to Seismic Restraint*, RP-812, American Society of Heating, Refrigerating, and Air-Conditioning Engineers, Atlanta, Ga.

American Society of Mechanical Engineers (ASME). (2010). "Standard for the seismic design and retrofit of above-ground piping systems." *B31E*, New York.

ANCO Engineers, Inc. (1983). "Seismic hazard assessment of non-structural components—Phase I." Final Report for National Science Foundation from ANCO Engineers, Inc., Culver City, Calif., September.

ASTM International. (2006). "Standard practice for application of ceiling suspension systems for acoustical tile and lay-in panels in areas requiring seismic restraint." *ASTM E580*, West Conshohocken, Penn.

ASTM International. (2007). "Standard specification for the manufacture, performance, and testing of metal suspension systems for acoustical tile and lay-in panel ceilings." *ASTM C635/C635M*, West Conshohocken, Penn.

ASTM International. (2008). "Standard practice for installation of metal ceiling suspension systems for acoustical tile and lay-in panels." *ASTM C636/C636M*, West Conshohocken, Penn.

ASTM International. (2010). "Standard practice for installation of ceiling suspension systems for acoustical tile and lay-in panels for areas subject to earthquake ground motion." *ASTM E580/E580M*, West Conshohocken, Penn.

Bachman, R. E., and Dowty, S. M. (2008). "Nonstructural component or nonbuilding structure?" *Building Safety Journal* (April–May).

Bouwkamp, J. G., and Meehan, J. F. (1960). "Drift limitations imposed by glass." *Proceedings of the Second World Conference on Earthquake Engineering*, Tokyo, Japan, 1763–1778.

Department of Defense (DOD). (2007). "Seismic design for buildings." *UFC 3-310-04*, U.S. Department of Defense, Unified Facilities Criteria, Washington, D.C.

FM Global. (2007). *Approved Standard for Pipe Couplings and Fittings for Aboveground Fire Protection Systems*. http://www.fmglobal.com/assets/pdf/fmapprovals/1920.pdf

Institute of Electrical and Electronics Engineers, Inc. (IEEE). (2005). "IEEE recommended practices for seismic design of substations" *IEEE 693-2005*, Piscataway, NJ.

International Code Council Evaluation Service. (2010). "Seismic qualification by shake-table testing of nonstructural components and systems." *ICC-ES AC156*, Whittier, Calif.

International Code Council Evaluation Service. (2012a). "Acceptance criteria for adhesive anchors in masonry elements." *ICC-ES AC58*, Whittier, Calif.

International Code Council Evaluation Service. (2012b). “Acceptance criteria for expansion anchors in masonry elements.” *ICC-ES AC01*, Whittier, Calif.
International Code Council Evaluation Service. (2012c). “Acceptance criteria for mechanical anchors in concrete elements.” *ICC-ES AC193*, Whittier, Calif.
International Code Council Evaluation Service. (2012d). “Acceptance criteria for post-installed adhesive anchors in concrete elements.” *ICC-ES AC 308*, Whittier, Calif.
International Code Council Evaluation Service. (2012e). “Acceptance criteria for predrilled fasteners (screw anchors) in masonry.” *ICC-ES AC106*, Whittier, Calif.
Kehoe, B., and Hachem, M. (2003). “Procedures for estimating floor accelerations.” ATC 29-2, *Proceedings of the Seminar on Seismic Design, Performance, and Retrofit of Nonstructural Components in Critical Facilities*, Newport Beach, Calif., 361–374.
Manufacturers Standardization Society of the Valve and Fitting Industry (MSS). (2001). “Bracing for piping systems seismic–wind–dynamic design, selection, application.” *MSS SP-127*, Vienna, Va.
National Earthquake Hazards Reduction Program (NEHRP). (2009). *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, National Earthquake Hazards Reduction Program, Washington, D.C.
National Fire Protection Association (NFPA). (2007). “Standard for the installation of sprinkler systems.” *NFPA 13*, Quincy, Mass.
National Fire Protection Association (NFPA). (2011). “National electric code (NEC).” *NFPA 70*, Quincy, Mass.
Sheet Glass Association of Japan. (1982). *Earthquake Safety Design of Windows*.
Sheet Metal and Air-Conditioning Contractors National Association (SMACNA). (2008). *Seismic Restraint Manual: Guidelines for Mechanical Systems*, 3rd ed., Sheet Metal and Air-Conditioning Contractors National Association, Chantilly, Va.
Structural Engineers Association of California (SEAOC). (1999). *Recommended Lateral Force Requirements and Commentary*, Structural Engineers Association of California, Sacramento, Calif.
Underwriter Laboratories (UL). (2004). “Rubber gasketed fittings for fire-protection service.” *UL 213*, Northbrook, Ill.
Wright, P. D. (1989). *The Development of a Procedure and Rig for Testing the Racking Resistance of Curtain Wall Glazing*, BRANZ Study Report 17, Building Research Association of New Zealand (BRANZ), Porirua, New Zealand.

OTHER REFERENCES (NOT CITED)

Bachman, R. E, and Drake, R. M. (1996). “A study to empirically validate the component response modification factors in the 1994 *NEHRP Provisions* design force equations for architectural, mechanical, and electrical components,” letter report to the National Center for Earthquake Engineering Research, July.
Bachman, R. E., Drake, R. M., and Richter, P. J. (1993). “1994 update to 1991 *NEHRP Provisions* for architectural, mechanical, and electrical components and systems,” letter report to the National Center for Earthquake Engineering Research, February 22.
Behr, R. A., and Belarbi, A. (1996). “Seismic tests methods for architectural glazing systems.” *Earthquake Spectra*, 12(1), 129–143.
Behr, R. A., Belarbi, A., and Brown, A. T. (1995). “Seismic performance of architectural glass in a storefront wall system.” *Earthquake Spectra*, 11(3), 367–391.
Drake, R. M., and Bachman, R. E. (1994). “1994 *NEHRP Provisions* for architectural, mechanical, and electrical components.” In *Proceedings of the 5th United States National Conference on Earthquake Engineering*.
Drake, R. M., and Bachman, R. E. (1995). “Interpretation of instrumented building seismic data and implications for building codes.” In *Proceedings of the 1995 SEAOC Annual Convention*.
Drake, R. M., and Bachman, R. E. (1996). “NEHRP provisions for 1994 for nonstructural components.” *J. Arch. Engrg.*, 2(1), 26–31.
Gates, W. E., and McGavin, G. (1998). “Lessons learned from the 1994 Northridge earthquake on the vulnerability of nonstructural systems.” In *Proceedings, Seminar on Seismic Design, Retrofit, and Performance of Nonstructural Components*, ATC-29-1, Applied Technology Council, Redwood City, Calif., 93–101.
Haroun, M. A. and Housner, G. W. (1981). “Seismic design of liquid storage tanks.” *J. Tech. Councils of ASCE*, 107(1), 191–207.
Pantelides, C. P., Truman, K. Z., Behr, R. A., and Belarbi, A. (1996). “Development of a loading history for seismic testing of architectural glass in a shop-front wall system.” *Engrg. Struct.*, 18(12), 917–935.

CHAPTER C14
MATERIAL SPECIFIC SEISMIC DESIGN AND DETAILING REQUIREMENTS

Because seismic loading is expected to cause nonlinear behavior in structures, seismic design criteria require not only provisions to govern loading but also provisions to define the required configurations, connections, and detailing to produce material and system behavior consistent with the design assumptions. Thus, although ASCE/SEI 7-10 is primarily a loading standard, compliance with Chapter 14, which covers material-specific seismic design and detailing, is required. In general, Chapter 14 adopts material design and detailing standards developed by material standards organizations. These material standards organizations maintain complete commentaries covering their standards, and such material is not duplicated here.

C14.0 SCOPE

The scoping statement in this section clarifies that foundation elements are subject to all of the structural design requirements of the standard.

C14.1 STEEL

C14.1.1 Reference Documents. This section lists a series of structural standards published by the American Institute of Steel Construction (AISC), the American Iron and Steel Institute (AISI), the American Society of Civil Engineers (ASCE/SEI), and the Steel Joist Institute (SJI), which are to be applied in the seismic design of steel members and connections in conjunction with the requirements of ASCE/SEI 7. The AISC references are available free of charge in electronic format at www.aisc.org, and the SJI references are available as a free download at www.steeljoist.org.

C14.1.2 Structural Steel

C14.1.2.1 General. This section adopts AISC 360 by direct reference. The specification applies to the design of the structural steel system or systems with structural steel acting compositely with reinforced concrete. In particular, the document sets forth criteria for the design, fabrication, and erection of structural steel buildings and other structures, where other structures are defined as structures designed, fabricated, and erected in a manner similar to buildings, with buildinglike vertical and lateral load-resisting elements. The document includes extensive commentary.

C14.1.2.2 Seismic Requirements for Structural Steel Structures

C14.1.2.2.1 Seismic Design Categories B and C. For the lower Seismic Design Categories (SDCs) B and C, a range of options are available in the design of a structural steel lateral force-resisting system. The first option is to design the structure to meet the design and detailing requirements in AISC 341 for structures assigned to higher SDCs, with the corresponding seismic design parameters (R, Ω_0, and C_d). The second option, presented in the exception, is to use an R factor of 3 (resulting in an increased base shear), an Ω_0 of 3, and a C_d value of 3 but without the specific seismic design and detailing required in AISC 341. The basic concept underlying this option is that design for a higher base shear force results in essentially elastic response that compensates for the limited ductility of the members and connections. The resulting performance is considered comparable to that of more ductile systems.

C14.1.2.2.2 Seismic Design Categories D through F. For the higher SDCs, the engineer must follow the seismic design provisions of AISC 341 using the seismic design parameters specified for the chosen structural system, except as permitted in Table 15.4-1. For systems other than those identified in Table 15.4-1, it is not considered appropriate to design structures without specific design and detailing for seismic response in these high SDCs.

C14.1.3 Cold-Formed Steel

C14.1.3.1 General. This section adopts two standards by direct reference: AISI S100, North American Specification for the Design of Cold-Formed Steel Structural Members, and ASCE/SEI 8, Specification for the Design of Cold Formed Stainless Steel Structural Members.

Both of the adopted reference documents have specific limits of applicability. AISI S100 (Section A1.1) applies to the design of structural members that are cold-formed to shape from carbon or low-alloy steel sheet, strip, plate, or bar not more than 1 in. thick. ASCE/SEI 8 (Section 1.1.1) governs the design of structural members that are cold-formed to shape from annealed and cold-rolled sheet, strip, plate, or flat bar stainless steels. Both documents focus on load-carrying members in buildings; however, allowances are made for applications in nonbuilding structures, if dynamic effects are considered appropriately.

Within each document, there are requirements related to general provisions for the applicable types of steel; design of elements, members, structural assemblies, connections, and joints; and mandatory testing. In addition, AISI S100 contains a chapter on the design of cold-formed steel structural members and connections undergoing cyclic loading. Both standards contain extensive commentaries.

C14.1.3.2 Seismic Requirements for Cold-Formed Steel Structures. This section adopts three standards by direct reference—AISI S100, ASCE 8, and AISI S110. Cold-formed steel and stainless steel members that are part of a seismic force-resisting system listed in Table 12.2-1 must be detailed in accordance with the appropriate standard: AISI S100 or ASCE 8.

Minimum Design Loads for Buildings and Other Structures

In addition, the section adopts a reference to AISI S110, which provides design provisions for a specific seismic force-resisting system entitled "cold-formed steel—special bolted moment frame" or CFS-SBMF. Sato and Uang (2007) have shown that this system experiences inelastic deformation at the bolted connections because of slip and bearing during significant seismic events. To develop the designated mechanism, requirements based on capacity design principles are provided for the design of the beams, columns, and associated connections. The document has specific requirements for the application of quality assurance and quality control procedures.

C14.1.4 Cold-Formed Steel Light-Frame Construction

C14.1.4.1 General. This subsection of cold-formed steel relates to light-framed construction, which is defined as a method of construction where the structural assemblies are formed primarily by a system of repetitive wood or cold-formed steel framing members or subassemblies of these members (Section 11.2 of this standard). It adopts Section D4 of AISI (2010), which directs the user to an additional suite of AISI standards, including AISI S200, AISI S210, AISI S211, AISI S212, AISI S213, and AISI S214.

In addition, all of these documents include commentaries to aid users in the correct application of their requirements.

C14.1.4.2 Seismic Requirements for Cold-Formed Steel Light-Frame Construction. Per AISI S213, Sections C1.1 and D1.1, all cold-formed steel light-frame construction systems with R greater than 3 must be designed in accordance with AISI S213 inclusive of Sections C5 and D3. In particular, this requirement includes all entries from Table 12-2.1 of this standard for "light-frame walls sheathed with wood structural panels … or steel sheets" and "light-frame wall systems using flat strap bracing."

Per AISI S213 Sections C1.1 and D1.1, cold-formed steel light-frame construction systems with R less than or equal to 3 are permitted to be designed and constructed exclusive of AISI S213, Sections C5 and D3 only—they must meet the other applicable requirements of AISI S213. This requirement includes entries from Table 12-2.1 of this standard for "light-frame walls with shear panels of all other materials" and "steel systems not specifically detailed for seismic resistance, excluding cantilever column systems."

C14.1.4.3 Prescriptive Cold-Formed Steel Light-Frame Construction. This section adopts AISI S230, *Standard for Cold-Formed Steel Framing – Prescriptive Method for One and Two Family Dwellings*, which applies to the construction of detached one- and two-family dwellings, townhouses, and other attached single-family dwellings not more than two stories in height using repetitive in-line framing practices (Section A1). This document adopts AISI S200 by direct reference and includes a commentary to aid the user in the correct application of its requirements.

C14.1.5 Steel Deck Diaphragms. Design of steel deck diaphragms is to be based on recognized national standards or a specific testing program directed by a person experienced in testing procedures and steel deck. All fastener design values (welds, screws, power-actuated fasteners, and button punches) for attaching steel deck sheet to steel deck sheet or for attaching the steel deck to the building framing members must be per recognized national design standards or specific steel deck testing programs. All steel deck diaphragm and fastener design properties must be approved for use by the authorities in whose jurisdiction the construction project occurs. Steel deck diaphragm in-plane design forces (seismic, wind, or gravity) must be determined per ASCE/SEI 7 Section 12.10.1. Steel deck manufacturer test reports prepared in accordance with this provision can be used where adopted and approved by the authority having jurisdiction for the building project. The diaphragm design manual produced by the Steel Deck Institute (2004) is also a reference for design values.

Steel deck is assumed to have a corrugated profile consisting of alternating up and down flutes that are manufactured in various widths and heights. Use of flat sheet metal as the overall floor or roof diaphragm is permissible where designed by engineering principles, but it is beyond the scope of this section. Flat or bent sheet metal may be used as closure pieces for small gaps or penetrations or for shear transfer over short distances in the steel deck diaphragm where diaphragm design forces are considered.

Steel deck diaphragm analysis must include design of chord members at the perimeter of the diaphragm and around interior openings in the diaphragm. Chord members may be steel beams attached to the underside of the steel deck designed for a combination of axial loads and bending moments caused by acting gravity and lateral loads.

Where diaphragm design loads exceed the bare steel deck diaphragm design capacity, then either horizontal steel trusses or a structurally designed concrete topping slab placed over the steel deck must be provided to distribute lateral forces. Where horizontal steel trusses are used, the steel deck must be designed to transfer diaphragm forces to the steel trusses. Where a structural concrete topping over the steel deck is used as the diaphragm, the diaphragm chord members at the perimeter of the diaphragm and edges of interior openings must be either (a) designed flexural reinforcing steel placed in the structural concrete topping or (b) steel beams located under the steel deck with connectors (that provide a positive connection) as required to transfer design shear forces between the concrete topping and steel beams.

C14.1.6 Steel Cables. These provisions reference ASCE/SEI 19-96, *Structural Applications of Steel Cables for Buildings*, for the determination of the design strength of steel cables.

C14.1.7 Additional Detailing Requirements for Steel Piles in Seismic Design Categories D through F. Steel piles used in higher SDCs are expected to yield just under the pile cap or foundation because of combined bending and axial load. Design and detailing requirements of AISC 341 for H-piles are intended to produce stable plastic hinge formation in the piles. Because piles can be subjected to tension caused by overturning moment, mechanical means to transfer such tension must be designed for the required tension force, but not less than 10% of the pile compression capacity.

C14.2 CONCRETE

The section adopts by reference ACI 318-11 for structural concrete design and construction. In addition, modifications to ACI 318-11 (2011) are made that are needed to coordinate the provisions of that material design standard with the provisions of ASCE/SEI 7. Work is ongoing to better coordinate the provisions of the two documents (ACI 318 [2011] and ASCE/SEI 7) such that the provisions in Section 14.2 will be significantly reduced in future editions of ASCE/SEI 7.

C14.2.2.1 Definitions. The first two definitions describe wall types for which definitions currently do not exist in ACI 318. These definitions are essential to the proper interpretation of the R and C_d factors for each wall type specified in Table 12.2-1.

C14.2.2.2 ACI 318 Section 7.10. Section 7.10.5.7 of ACI 318 prescribes reinforcement details for ties in compression members. This modification prescribes additional details for ties around anchor bolts in structures assigned to SDCs C through F.

C14.2.2.3 Scope. This provision describes how the ACI 318 provisions should be interpreted for consistency with the ASCE 7 provisions.

C14.2.2.4 Intermediate Precast Structural Walls. Section 21.4 of ACI 318 imposes requirements on precast walls for moderate seismic risk applications. Ductile behavior is to be ensured by yielding of the steel elements or reinforcement between panels or between panels and foundations. This provision requires the designer to determine the deformation in the connection corresponding to the earthquake design displacement and then to check from experimental data that the connection type used can accommodate that deformation without significant strength degradation.

Several steel element connections have been tested under simulated seismic loading, and the adequacy of their load-deformation characteristics and strain capacity have been demonstrated (Schultz and Magana 1996). One such connection was used in the five-story building test that was part of the PRESSS Phase 3 research. The connection was used to provide damping and energy dissipation, and it demonstrated a very large strain capacity (Nakaki et al. 2001). Since then, several other steel element connections have been developed that can achieve similar results (Banks and Stanton 2005, Nakaki et al. 2005). In view of these results, it is appropriate to allow yielding in steel elements that have been shown experimentally to have adequate strain capacity to maintain at least 80% of their yield force through the full design displacement of the structure.

C14.2.2.6 Foundations. The intention is that there should be no conflicts between the provisions of Section 21.12 of ACI 318 and Sections 12.1.5, 12.13, and 14.2 of ASCE/SEI 7. However, the additional detailing requirements for concrete piles of Section 14.2.3 can result in conflicts with ACI 318 provisions if the pile is not fully embedded in the soil.

C14.2.2.7 Detailed Plain Concrete Shear Walls. Design requirements for plain masonry walls have existed for many years, and the corresponding type of concrete construction is the plain concrete wall. To allow the use of such walls as the lateral force-resisting system in SDCs A and B, this provision requires such walls to contain at least the minimal reinforcement specified in Section 22.6.7.2.

C14.2.3 Additional Detailing Requirements for Concrete Piles. Chapter 20 of PCI (2004) provides detailed information on the structural design of piles and on pile-to-cap connections for precast prestressed concrete piles. ACI 318 does not contain provisions governing the design and installation of portions of concrete piles, drilled piers, and caissons embedded in ground except for SDC D, E, and F structures.

C14.2.3.1.2 Reinforcement for Uncased Concrete Piles (SDC C). The transverse reinforcing requirements in the potential plastic hinge zones of uncased concrete piles in SDC C are a selective composite of two ACI 318 requirements. In the potential plastic hinge region of an intermediate moment-resisting concrete frame column, the transverse reinforcement spacing is restricted to the least of (1) eight times the diameter of the smallest longitudinal bar, (2) 24 times the diameter of the tie bar, (3) one-half the smallest cross-sectional dimension of the column, and (4) 12 in. Outside of the potential plastic hinge region of a special moment-resisting frame column, the transverse reinforcement spacing is restricted to the smaller of six times the diameter of the longitudinal column bars and 6 in.

C14.2.3.1.5 Reinforcement for Precast Nonprestressed Concrete Piles (SDC C). Transverse reinforcement requirements in and outside of the plastic hinge zone of precast nonprestressed piles are clarified. The transverse reinforcement requirement in the potential plastic hinge zone is a composite of two ACI 318 requirements (see Section C14.2.3.1.2). Outside of the potential plastic hinge region, the transverse reinforcement spacing is restricted to 16 times the longitudinal bar diameter. This restriction should permit the longitudinal bars to reach compression yield before buckling. The maximum 8-in. tie spacing comes from current building code provisions for precast concrete piles.

C14.2.3.1.6 Reinforcement for Precast Prestressed Piles (SDC C). The transverse and longitudinal reinforcing requirements given in ACI 318, Chapter 21, were never intended for slender precast prestressed concrete elements and will result in unbuildable piles. These requirements are based on PCI Committee on Prestressed Concrete Piling (1993).

Equation 14.2-1, originally from ACI 318, has always been intended to be a lower-bound spiral reinforcement ratio for larger diameter columns. It is independent of the member section properties and can therefore be applied to large- or small-diameter piles. For cast-in-place concrete piles and precast prestressed concrete piles, the spiral reinforcing ratios resulting from this formula are considered to be sufficient to provide moderate ductility capacities (Fanous et al. 2007).

Full confinement per Eq. 14.2-1 is required for the upper 20 ft of the pile length where curvatures are large. The amount is relaxed by 50% outside of that length in view of lower curvatures and in consideration of confinement provided by the soil.

C14.2.3.2.3 Reinforcement for Uncased Concrete Piles (SDC D through F). The reinforcement requirements for uncased concrete piles are taken from current building code requirements and are intended to provide ductility in the potential plastic hinge zones (Fanous et al. 2007).

C14.2.3.2.5 Reinforcement for Precast Concrete Piles (SDC D through F). The transverse reinforcement requirements for precast nonprestressed concrete piles are taken from the IBC (2012) requirements and should be adequate to provide ductility in the potential plastic hinge zones (Fanous et al. 2007).

C14.2.3.2.6 Reinforcement for Precast Prestressed Piles (SDC D through F). The reduced amounts of transverse reinforcement specified in this provision compared with those required for special moment frame columns in ACI 318 are justified by the results of the study by Fanous et al. (2007). The last paragraph provides minimum transverse reinforcement outside of the zone of prescribed ductile reinforcing.

C14.3 COMPOSITE STEEL AND CONCRETE STRUCTURES

This section provides guidance on the design of composite and hybrid steel-concrete structures. Composite structures are defined as those incorporating structural elements made of steel and concrete portions connected integrally throughout the structural element by mechanical connectors, bond, or both. Hybrid structures are defined as consisting of steel and concrete structural elements connected together at discrete points. Composite and hybrid structural systems mimic many of the existing steel (moment and braced frame) and reinforced concrete (moment

frame and wall) configurations but are given their own design coefficients and factors in Table 12.2-1. Their design is based on ductility and energy dissipation concepts comparable to those used in conventional steel and reinforced concrete structures, but it requires special attention to the interaction of the two materials because it affects the stiffness, strength, and inelastic behavior of the members, connections, and systems.

C14.3.1 Reference Documents. Seismic design for composite structures assigned to SDCs D, E, or F is governed primarily by ANSI/AISC 341. Composite design provisions in ANSI/AISC 341 are less prescriptive than those for structural steel and provide flexibility for designers to use analytical tools and results of research in their practice. Composite structures assigned to SDC A, B, or C may be designed according to principles outlined in ANSI/AISC 360 and ACI 318. ANSI/AISC 360 and ACI 318 provide little guidance on connection design; therefore, designers are encouraged to review ANSI/AISC 341 for guidance on the design of joint areas. Differences between older AISC and ACI provisions for cross-sectional strength for composite beam-columns have been minimized by changes in the latest ANSI/AISC 360, and ANSI/AISC 360 refers to ACI 318 for much of the design of reinforced concrete components of composite structures. However, there is not uniform agreement between the provisions in ACI 318 and ANSI/AISC 360 regarding detailing, limits on material strengths, stability, and strength for composite beam-columns. The composite design provisions in ANSI/AISC 360 are considered to be current.

C14.3.4 Metal-Cased Concrete Piles. Design of metal-cased concrete piles, which are analogous to circular concrete filled tubes, is governed by Sections 14.2.3.1.3 and 14.2.3.2.4 of this standard. The intent of these provisions is to require metal-cased concrete piles to have confinement and protection against long-term deterioration comparable to that for uncased concrete piles.

C14.4 MASONRY

This section adopts by reference and then makes modifications to TMS 402/ACI 530/ASCE 5 and TMS 602/ACI 530.1/ASCE 6, which are commonly referred to as the "MSJC Standards (Code and Specification)" after the Masonry Standards Joint Committee (MSJC), which is charged with development and maintenance of these standards. In past editions of this standard, modifications to these referenced standards were made. During the development of the 2008 edition of the MSJC standards, each of these modifications was considered by the MSJC. Some were incorporated directly into the MSJC standards. These modifications have accordingly been removed from the modifications in this standard. Work is ongoing to better coordinate the provisions of the two documents (MSJC and ASCE/SEI 7) so that the provisions in Section 14.4 are significantly reduced or eliminated in future editions.

C14.5 WOOD

C14.5.1 Reference Documents. Two national consensus standards are adopted for seismic design of engineered wood structures: the *National Design Specification* (NDS), and the *Special Design Provisions for Wind and Seismic* (SDPWS). Both of these standards are presented in dual allowable stress design (ASD) and load and resistance factor design (LRFD) formats. Both standards reference a number of secondary standards for related items such as wood materials and fasteners. SDPWS addresses general principles and specific detailing requirements for shear wall and diaphragm design and provides tabulated nominal unit shear capacities for shear wall and diaphragm sheathing and fastening. The balance of member and connection design is to be in accordance with the NDS.

REFERENCES

American Concrete Institute (ACI). (2008). "Building code requirements for structural concrete and commentary," *ACI 318*, Farmington Hills, MI.

American Concrete Institute (ACI). (2011). "Building code requirements for structural concrete and commentary," *ACI 318*, Farmington Hills, MI.

American Institute of Steel Construction (AISC). (2010a). "Seismic provisions for structural steel buildings." *AISC 341*, Chicago.

American Institute of Steel Construction (AISC). (2010b). "Specification for structural steel buildings. *AISC 360*, Chicago.

American Institute of Timber Construction. (2005). *Timber Construction Manual*, 5th Ed. John Wiley and Sons.

American Iron and Steel Institute (AISI). (2007a). "North American standard for cold-formed steel framing—General provisions." *S200*, Washington, D.C.

American Iron and Steel Institute (AISI). (2007b). "North American standard for cold-formed steel framing—Floor and roof system design," *S210*, Washington, D.C.

American Iron and Steel Institute (AISI). (2007c). "North American standard for cold-formed steel framing—Wall stud design." *S211*, Washington, D.C.

American Iron and Steel Institute (AISI). (2007d). "North American standard for cold-formed steel framing—Header design." *S212*, Washington, D.C.

American Iron and Steel Institute (AISI). (2007e). "North American standard for cold-formed steel framing—Lateral design." *S213*, Washington, D.C.

American Iron and Steel Institute (AISI). (2007f). "North American standard for cold-formed steel framing—Supplement 2, truss design." *S214*, Washington, D.C.

American Iron and Steel Institute (AISI). (2007g). "Standard for cold-formed steel framing—Prescriptive method for one and two family dwellings." *S230*, Washington, D.C.

American Iron and Steel Institute (AISI). (2009). "Standard for seismic design of cold-formed steel structural systems—Special bolted moment frames." *ANSI/AISI S110 w/S1-09,* 2007, with Supplement 1, Washington, D.C.

American Iron and Steel Institute (AISI). (2010), "North American Specification for the design of cold-formed steel structural members." *ANSI/AISI S100 w/S2-10*, 2007, with Supplement 2, Washington, D.C.

American Society of Civil Engineers (ASCE). (2002). "Specification for the design of cold-formed stainless steel structural members." *ASCE 8-02*. Reston, Va.

American Society of Civil Engineers (ASCE). (2010). "Structural applications of steel cables for buildings." *ASCE 19-10*, Reston, Va.

American Society of Civil Engineers (ASCE). (2011). "Building code requirements and specifications for masonry structures." *TMS 402-11/CA 530-11/ASCE 5-11*, Reston, Va.

American Wood Council (AWC). (2008). "Special design provisions for wind and seismic (wind & seismic)." *ANSI/AWC SDPWS-2012*, Leesburg, Va.

American Wood Council (AWC). (2012). "National Design Specification (NDS) for wood construction with commentary." *ANSI/AWC NDS-2012*, Leesburg, Va.

APA, The Engineered Wood Association. (1994). "Northridge California earthquake." *T94-5*, Tacoma, Wash.

APA, The Engineered Wood Association. (2004). "Diaphragms and shear walls design/construction guide." *L350*, Tacoma, Wash.

Applied Technology Council. (1981). "Guidelines for the design of horizontal wood diaphragms." *ATC-7*, Redwood City, Calif.

Banks, G., and Stanton, J. (2005). "*Panel-to-panel connections for hollow-core shear walls subjected to seismic loading*." In *Proceedings, 2005 PCI Convention*, Precast/Prestressed Concrete Institute, Chicago.

Bora, C., Oliva, M. G., Nakaki, S. D., and Becker, R. (2007). "Development of a precast concrete shear-wall system requiring special code acceptance." *PCI J.*, 52(1), 122–135.

Breyer et al. (2006). *Design of Wood Structures ASD/LRFD*, Sixth Ed. McGraw-Hill Book Company, New York.

Canadian Wood Council. (1995). *Wood Reference Handbook*. Canadian Wood Council, Ottawa.

Canadian Wood Council. (2005). *Wood Design Manual 2005*. Canadian Wood Council, Ottawa.

Cobeen, K. (2004). "Recent developments in the seismic design and construction of woodframe buildings." Chapter 18, *Earthquake Engineering from Engineering Seismology to Performance-Based Engineering*, Yousef Bozorgia and Vitelmo Bertero, eds., CRC Press, Boca Raton, Fla.

Consortium of Universities for Research in Earthquake Engineering (CUREE). (2004). *Recommendations for Earthquake Resistance in the Design and Construction of Woodframe Buildings*, CUREE W-30. CUREE, Richmond, Calif.

Department of the Army, Navy, and Air Force. (1992). "Seismic design for buildings." *TM5-809-10* (Tri-Services Manual). U.S. Government Printing Office, Washington, D.C.

Dolan, J. D. (2003). "Wood structures." Chapter 15, *Earthquake Engineering Handbook*, Wai-Fah Chen and Charles Scawthorn, eds., CRC Press, Boca Raton, Fla.

Earthquake Engineering Research Institute. (1996). "Northridge earthquake reconnaissance report." *Earthquake Spectra*, Chapter 6 of Supplement C to Volume 11, EERI, Oakland, CA.

Faherty, Keith F., and Williamson, T. G. (1989). *Wood Engineering and Construction Handbook*. McGraw-Hill, New York.

Fanous, A., Sritharan, S., Suleiman, M., and Arulmoli, A. (2007). *Minimum spiral reinforcement requirements and lateral displacement limits for prestressed concrete piles in high seismic regions*, Department of Civil, Construction and Environmental Engineering, Iowa State University, ISU-ERI Ames Report.

Federal Emergency Management Agency (FEMA). (2003). "NEHRP recommended provisions for seismic regulations for new buildings and other structures." *FEMA 450*, Federal Emergency Management Agency, Building Seismic Safety Council, Washington, D.C.

Federal Emergency Management Agency. (2005). *Coastal Construction Manual*, Third Ed., FEMA 55. FEMA, Washington, D.C.

Forest Products Laboratory. (1986). *Wood: Engineering Design Concepts*. Materials Education Council, The Pennsylvania State University, University Park.

Goetz, K. H., Hoor, D., Moehler, K., and Natterer, J. (1989). *Timber Design and Construction Source Book: A Comprehensive Guide to Methods and Practice*. McGraw-Hill, New York.

Hoyle, R. J., and Woeste, F. E. (1989). *Wood Technology in the Design of Structures*. Iowa State University Press, Ames.

International Code Council. (2006). "ICC standard on the design and construction of log structures," Third Draft. Country Club Hills, Ill.

International Code Council. (2012). "International building code." Country Club Hills, IL.

Ishizuka, T., and Hawkins, N. M. (1987). *Effect of Bond Deterioration on the Seismic Response of Reinforced and Partially Prestressed Concrete Ductile Moment Resistant Frames*, Report SM 87-2. Department of Civil Engineering, University of Washington, Seattle.

Karacabeyli, E., and Popovsky, M. (2003). "Design for earthquake resistance." Chapter 15 of *Timber Engineering*, H. Larsen and S. Thelandersson, eds. John Wiley & Sons, New York.

Keenan, F. J. (1986). *Limit States Design of Wood Structures*. Morrison Hershfield Limited, North York, ON.

Lee, N. H., Kim, K. S., Bang, C. J., and Park, K. R. (2007). "Tensile-headed anchors with large diameter and deep embedment in concrete." *ACI Struct. J.*, 104(4), 479–486.

Lee, N. H., Park, K. R., and Suh, Y. P. (2010). "Shear behavior of headed anchors with large diameters and deep embedments." *ACI Struct. J.*, 107 (2), 146–156.

Masonry Standards Joint Committee (MSJC). (2005a). "Building code requirements for masonry structures." *ACI 530-05/ASCE/SEI 5-05/TMS 402-5*, Farmington Hills, MI.

Masonry Standards Joint Committee. (2005b). "Specification for masonry structures." *ACI 530.1-05/ASCE/SEI 6-05/TMS 602-05*, Farmington Hills, MI.

Masonry Standards Joint Committee (MSJC). (1999). "Building code requirements for masonry structures." *ACI 530-99/ASCE 5-99/TMS 402-99*, "Specification for masonry structures." *ACI 530.1-99/ASCE 6-99/TMS 602-99*, "Commentary on building code requirements for masonry structures, commentary on specification for masonry structures," known as "MSJC Standards (Code and Specification)," The Masonry Society, Boulder, Colo., American Concrete Institute, Farmington Hills, Mich., American Society of Civil Engineers, Reston, Va.

Nakaki, S., Becker, R., Oliva, M. G., and Paxson, D. (2005). "New connections for precast wall systems in high seismic regions." In *Proceedings, 2005 PCI Convention*, Precast/Prestressed Concrete Institute, Chicago.

Nakaki, S., Stanton, J. F., and Sritharan, S., (2001) "The PRESSS Five-Story Precast Concrete Test Building, University of California, San Diego, La Jolla, California," *PCI Journal*, 46(5), 20–26.

Nakaki, S. D., Stanton, J. F., and Sritharan, S. (1999). "An overview of the PRESSS five-story precast test building." *PCI J.*, 44(2), 26–39.

Park, R., and Thompson, K. J.. (1977). "Cyclic load tests on pretsressed and partially prestressed beam-column joints." *PCI J.*, 22(5), 84–110.

PCI Committee on Prestressed Concrete Piling. (1993). "Recommended practice for design, manufacture and installation of prestressed concrete piling." *PCI J.*, 38(2), 14–41.

Precast/Prestressed Concrete Institute (PCI). (2004). "Precast prestressed concrete piles." Chapter 20, *Bridge Design Manual*, BM-20-04. PCI, Chicago.

Sato, A., and Uang, C. M. (2007). "Development of a seismic design procedure for cold-formed steel bolted frames," Report No. SSRP-07/16, University of California, San Diego.

Schultz, A. E., and Magana, R. A. (1996). "Seismic behavior of connections in precast concrete walls." *Proceedings, Mete A. Sozen Symposium*, SP-162, American Concrete Institute, Farmington Hills, Mich., 273–311.

Sherwood, G. E., and Stroh, R. C. (1989). "Wood-frame house construction." In *Agricultural Handbook 73*, U.S. Government Printing Office, Washington, D.C.

Somayaji, Shan. (1992). *Structural Wood Design*. West Publishing Co., St. Paul, Minn.

Stalnaker, J. J., and Harris, E. C. (1996). *Structural Design in Wood*, 2nd Ed., McGraw-Hill, New York.

Steel Deck Institute. (2004). *Diaphragm Design Manual*, 3rd Ed., No. DDMO3. SDI, Fox River Grove, Ill.

Structural Engineeers Association of California. (1999). *Recommended Lateral Force Requirements and Commentary*. SEAOC, Sacramento, Calif.

Structural Engineeers Association of Northern California. (2005). *Guidelines for Seismic Evaluation and Rehabilitation of Tilt-Up Buildings and Other Rigid Wall/Flexible Diaphragm Structures*. SEAONC, Sacramento, Calif.

U.S. Department of Agriculture, National Oceanic and Atmospheric Administration. (1971). *San Fernando, California, Earthquake of February 9, 1971*. NOAA, Washington, D.C.

Minimum Design Loads for Buildings and Other Structures

CHAPTER C15
SEISMIC DESIGN REQUIREMENTS FOR NONBUILDING STRUCTURES

C15.1 GENERAL

C15.1.1 Nonbuilding Structures. Building codes traditionally have been perceived as minimum standards for the design of nonbuilding structures, and building code compliance of these structures is required by building officials in many jurisdictions. However, requirements in the industry reference documents are often at odds with building code requirements. In some cases, the industry documents need to be altered, while in other cases, the building codes need to be modified. Registered design professionals are not always aware of the numerous accepted documents within an industry and may not know whether the accepted documents are adequate. One of the intents of Chapter 15 of the standard is to bridge the gap between building codes and existing industry reference documents.

Differences between the ASCE/SEI 7-10 design approaches for buildings and industry document requirements for steel multilegged water towers (Figure C15.1-1) are representative of this inconsistency. Historically, such towers have performed well when properly designed in accordance with American Water Works Association (AWWA) standards and industry practices. Those standards and practices differ from the ASCE/SEI 7-10 treatment of buildings in that tension-only rods are allowed, upset rods are preloaded at the time of installation, and connection forces are not amplified.

Chapter 15 also provides an appropriate link so that the industry reference documents can be used with the seismic ground motions established in the standard. Some nonbuilding structures are similar to buildings and can be designed using sections of the standard directly, whereas other nonbuilding structures require special analysis unique to the particular type of nonbuilding structure.

Building structures, vehicular bridges, electrical transmission towers, hydraulic structures (e.g., dams), buried utility lines and their appurtenances, and nuclear reactors are excluded from the scope of the nonbuilding structure requirements, although industrial buildings are permitted per Chapter 11 to use the provisions in Chapter 15 for nonbuilding structures with structural systems similar to buildings, provided specific conditions are met. The excluded structures are covered by other well established design criteria (e.g., electrical transmission towers and vehicular bridges) are not under the jurisdiction of local building officials (e.g., nuclear reactors and dams), or require technical considerations beyond the scope of the standard (e.g., buried utility lines and their appurtenances).

C15.1.2 Design. Nonbuilding structures and building structures have much in common with respect to design intent and expected performance, but there are also important differences. Chapter 15 relies on other portions of the standard where possible and provides special notes where necessary.

There are two types of nonbuilding structures: those with structural systems similar to buildings and those with structural systems not similar to buildings. Specific requirements for these two cases appear in Sections 15.5 and 15.6.

C15.1.3 Structural Analysis Procedure Selection. Nonbuilding structures that are similar to buildings are subject to the same analysis procedure limitations as building structures. Nonbuilding structures that are not similar to buildings are subject to those limitations and are subject to procedure limitations prescribed in applicable specific reference documents.

For many nonbuilding structures supporting flexible system components, such as pipe racks (Fig. C15.1-2), the supported piping and platforms generally are not regarded as rigid enough to redistribute seismic forces to the supporting frames.

For nonbuilding structures supporting very stiff (i.e., rigid) system components, such as steam turbine generators (STGs) and heat recovery steam generators (HRSGs) (Fig. C15.1-3), the supported equipment, ductwork, and other components (depending on how they are attached to the structure) may be rigid enough to redistribute seismic forces to the supporting frames. Torsional effects may need to be considered in such situations.

Section 12.6 presents seismic analysis procedures for building structures based on the seismic design category (SDC); the fundamental period, T; and the presence of certain horizontal or vertical irregularities in the structural system. Where the fundamental period is greater than or equal to 3.5 T_s (where $T_s = S_{D1}/S_{DS}$), the use of the equivalent lateral force procedure is not permitted in SDCs D, E, and F. This requirement is based on the fact that, unlike the dominance of the first mode response in case of buildings with lower first mode period, higher vibration modes do contribute more significantly in situations when the first mode period is larger than 3.5 T_s. For buildings that exhibit classic flexural deformation patterns (such as slender shear-wall or braced-frame systems), the second mode frequency is at least 3.5 times the first mode frequency, so where the fundamental period exceeds 3.5 T_s, the higher modes have larger contributions to the total response because they occur near the peak of the design response spectrum.

It follows that dynamic analysis (modal response spectrum analysis or response-history analysis) may be necessary to properly evaluate buildinglike nonbuilding structures if the first mode period is larger than 3.5 T_s and the equivalent lateral force analysis is sufficient for nonbuilding structures that respond as single-degree-of-freedom systems.

The recommendations for nonbuilding structures provided in the following are intended to supplement the designer's judgment and experience. The designer is given considerable latitude in selecting a suitable analysis method for nonbuilding structures.

Buildinglike Nonbuilding Structures. Table 12.6-1 is used in selecting analysis methods for buildinglike nonbuilding

structures, but, as illustrated in the following three conditions, the relevance of key behavior must be considered carefully:

1. Irregularities: Table 12.6-1 requires dynamic analysis for SDC D, E, and F structures that have certain horizontal or vertical irregularities. Some of these building irregularities (defined in Section 12.3.2) are relevant to nonbuilding structures. The weak-and soft-story vertical irregularities (Types 1a, 1b, 5a, and 5b of Table 12.3-2) are pertinent to the behavior of buildinglike nonbuilding structures. Other vertical and horizontal irregularities may or may not be relevant, as described below.
   a. Horizontal irregularities: Horizontal irregularities of Type 1a and 1b affect the choice of analysis method, but these irregularities apply only where diaphragms are rigid or semirigid, and some buildinglike nonbuilding structures have either no diaphragms or flexible diaphragms.
   b. Vertical irregularities: Vertical irregularity Type 2 is relevant where the various levels actually support significant loads. Where a buildinglike nonbuilding structure supports significant mass at a single level while other levels support small masses associated with stair landings, access platforms, and so forth, dynamic response is dominated by the first mode, so the equivalent lateral force procedure may be applied. Vertical irregularity Type 3 addresses large differences in the horizontal dimension of the seismic force-resisting system in adjacent stories, because the resulting stiffness distribution can produce a fundamental mode shape unlike that assumed in the development of the equivalent lateral force procedure. Because the concern relates to stiffness distribution, the horizontal dimension of the seismic force-resisting system, not of the overall structure, is important.
2. Arrangement of supported masses: Even where a nonbuilding structure has buildinglike appearance, it may not behave like a building, depending on how masses are attached. For example, the response of nonbuilding structures with suspended vessels and boilers cannot be determined reliably using the equivalent lateral force procedure because of the pendulum modes associated with the significant mass of the suspended components. The resulting pendulum modes, while potentially reducing story shears and base shear, may require large clearances to allow pendulum motion of the supported components and may produce excessive demands on attached piping. Dynamic analysis is highly recommended in such cases, with consideration for appropriate impact forces in the absence of adequate clearances.
3. Relative rigidity of beams: Even where a classic building model may seem appropriate, the equivalent lateral force procedure may underpredict the total response if the beams are flexible relative to the columns (of moment frames) or the braces (of braced frames). This underprediction occurs because higher modes associated with beam flexure may contribute more significantly to the total response (even if the first mode response is at a period less than 3.5 T_s). This situation of flexible beams can be especially pronounced for nonbuilding structures because the "normal" floors common to buildings may be absent. Therefore, the dynamic analysis procedures are suggested for buildinglike nonbuilding structures with flexible beams.

Nonbuilding Structures Not Similar to Buildings. The (static) equivalent lateral force procedure is based on classic building dynamic behavior, which differs from the behavior of many nonbuilding structures not similar to buildings. As discussed below, several issues should be considered for selecting either an appropriate method of dynamic analysis or a suitable distribution of lateral forces for static analysis.

1. Structural geometry: The dynamic response of nonbuilding structures with a fixed base and a relatively uniform distribution of mass and stiffness, such as bottom-supported vertical vessels, stacks, and chimneys, can be represented adequately by a cantilever (shear building) model. For these structures, the equivalent lateral force procedure provided in the standard is suitable. This procedure treats the dynamic response as being dominated by the first mode. In such cases, it is necessary to identify the first mode shape (using, for instance, the Rayleigh–Ritz method or other classical methods from the literature) for distribution of the dynamic forces. For some structures, such as tanks with low height-to-diameter ratios storing granular solids, it is conservative to assume a uniform distribution of forces. Dynamic analysis is recommended for structures that have neither a uniform distribution of mass and stiffness nor an easily determined first mode shape.
2. Number of lateral supports: Cantilever models are obviously unsuitable for structures with multiple supports. Figure C15.1-4 shows a nonbuilding braced frame structure that provides nonuniform horizontal support to a piece of equipment. In such cases, the analysis should include coupled model effects. For such structures, an application of the equivalent lateral force method could be used, depending on the number and locations of the supports. For example, most beam-type configurations lend themselves to application of the equivalent lateral force method.
3. Method of supporting dead weight: Certain nonbuilding structures (such as power boilers) are supported from the top. They may be idealized as pendulums with uniform mass distribution. In contrast, a suspended platform may be idealized as a classic pendulum with concentrated mass. In either case, these types of nonbuilding structures can be analyzed adequately using the equivalent lateral force method by calculating the appropriate frequency and mode shape. Figure C15.1-5 shows a nonbuilding structure containing lug-supported equipment with W_P greater than $0.25(W_S + W_P)$. In such cases, the analysis should include a coupled system with the mass of the equipment and the local flexibility of the supports considered in the model. Where the support is located near the nonbuilding structure's vertical location of the center of mass, a dynamic analysis is recommended.
4. Mass irregularities: Just as in the case of buildinglike nonbuilding structures, the presence of significantly uneven mass distribution is a situation where the equivalent lateral force method is not likely to provide a very accurate and perhaps unconservative force distribution. The dynamic analysis methods are recommended in such situations. Figure C15.1-6 illustrates two such situations. In part (a), a mass irregularity exists if W_1 is greater than 1.5 W_2 or less than 0.67 W_2. In part (b), a mass irregularity exists if W_3 is greater than either 1.5 W_2 or 1.5 W_4.
5. Torsional irregularities: Structures in which the fundamental mode of response is torsional or in which modes with significant mass participation exhibit a prominent torsional component may also have inertial force distributions that are significantly different from that predicted by the equivalent lateral force method. In such cases, dynamic analyses

should be considered. Figure C15.1-7 illustrates one such case where a vertical vessel is attached to a secondary vessel with W_2 greater than about $0.25(W_1 + W_2)$.

6. Stiffness and strength irregularities: Just as for buildinglike nonbuilding structures, abrupt changes in the distribution of stiffness or strength in a nonbuilding structure not similar to buildings can result in substantially different inertial forces from those indicated by the equivalent lateral force method. Figure C15.1-8 represents one such case. For structures having such configurations, consideration should be given to use of dynamic analysis procedures. Even where dynamic analysis is required, the standard does not define in any detail the degree of modeling; an adequate model may have a few dynamic degrees of freedom or tens of thousands of dynamic degrees of freedom. The important point is that the model captures the significant dynamic response features so that the resulting lateral force distribution is valid for design. The designer is responsible for determining whether dynamic analysis is warranted and, if so, the degree of detail required to address adequately the seismic performance.
7. Coupled response: Where the weight of the supported structure is large compared with the weight of the supporting structure, the combined response can be affected significantly by the flexibility of the supported nonbuilding structure. In that case, dynamic analysis of the coupled system is recommended. Examples of such structures are shown in Fig. C15.1-9. Part (a) shows a flexible nonbuilding structure with W_p greater than $0.25(W_s + W_p)$, supported by a relatively flexible structure; the flexibility of the supports and attachments should be considered. Part (b) shows flexible equipment connected by a large-diameter, thick-walled pipe and supported by a flexible structure; the structures should be modeled as a coupled system including the pipe.

C15.2 REFERENCE DOCUMENTS

Chapter 15 of the standard makes extensive use of reference documents in the design of nonbuilding structures for seismic forces. The documents referenced in Chapter 15 are industry documents commonly used to design specific types of nonbuilding structures. The vast majority of these reference documents contain seismic provisions that are based on the seismic ground motions of the 1997 UBC or earlier editions of the UBC. To use these reference documents, Chapter 15 modifies the seismic force provisions of these reference documents through the use of "bridging equations." The standard only modifies industry documents that specify seismic demand and capacity. The bridging equations are intended to be used directly with the other provisions of the specific reference documents. Unlike the other provisions of the standard, if the reference documents are written in terms of allowable stress design, then the bridging equations are shown in allowable stress design format. In addition, the detailing requirements referenced in Tables 15.4-1 and Table 15.4-2 are expected to be followed, as well as the general requirements found in Section 15.4.1. The usage of reference documents in conjunction with the requirements of Section 15.4.1 are summarized below in Table C15.2-1.

Currently, only four reference documents have been revised to meet the seismic requirements of the standard. AWWA D100-05, API 620 11th Edition Addendum 1 (2009), API 650 11th Edition Addendum 1 (2008), and ANSI/MH 16.1 (2008) have been adopted by reference in the standard without modification, except that height limits are imposed on "elevated tanks on symmetrically braced legs (not similar to buildings)" in AWWA D100-05 and the anchorage requirements of Section 15.4.9 are imposed on steel storage racks in ANSI/MH 16.1 (2008). Three of these reference documents apply to welded steel liquid storage tanks.

C15.3 NONBUILDING STRUCTURES SUPPORTED BY OTHER STRUCTURES

There are instances where nonbuilding structures not similar to buildings are supported by other structures or other nonbuilding structures. This section specifies how the seismic design loads for such structures are to be determined and the detailing requirements that are to be satisfied in the design.

C15.3.1 Less than 25% Combined Weight Condition. In many instances, the weight of the supported nonbuilding structure is relatively small compared with the weight of the supporting structure, such that the supported nonbuilding structure has a relatively small effect on the overall nonlinear earthquake response of the primary structure during design-level ground motions. It is permitted to treat such structures as nonstructural components and use the requirements of Chapter 13 for their design. The ratio of secondary component weight to total weight of 25% at which this treatment is permitted is based on judgment and was introduced into code provisions in the 1988 *Uniform Building Code* by the SEAOC Seismology Committee. Analytical studies, typically based on linear elastic primary and

Table C15.2-1 Usage of Reference Documents in Conjunction with Section 15.4.1

Subject	Requirement
R, Ω_0, and C_d values, detailing requirements, and height limits	Use values and limits in Tables 12.2-1, 15.4-1, or 15.4-2 as appropriate. Values from the reference document are not to be used.
Minimum base shear	Use the appropriate value from Eq. 15.4-1 or 15.4-2 for nonbuilding structures not similar to buildings. For structures containing liquids, gases, and granular solids supported at the base, the minimum seismic force cannot be less than that required by the reference document.
Importance factor	Use the value from Section 15.4.1.1 based on risk category. Importance factors from the reference document are not to be used unless they are greater than those provided in the standard.
Vertical distribution of lateral load	Use requirements of Section 12.8.3 or Section 12.9 or the applicable reference document.
Seismic provisions of reference documents	The seismic force provisions of reference documents may be used only if they have the same basis as Section 11.4 and the resulting values for total lateral force and total overturning moment are no less than 80% of the values obtained from the standard.
Load combinations	Load combinations specified in Section 2.3 (LRFD) or Section 15 (includes ASD load combinations of Section 2.4) must be used.

secondary structures, indicate that the ratio should be lower, but the SEAOC Seismology Committee judged that the 25% ratio is appropriate where primary and secondary structures exhibit nonlinear behavior that tends to lessen the effects of resonance and interaction. In cases where a nonbuilding structure (or nonstructural component) is supported by another structure, it may be appropriate to analyze in a single model. In such cases, it is intended that seismic design loads and detailing requirements be determined following the procedures of Section 15.3.2. Where there are multiple large nonbuilding structures, such as vessels supported on a primary nonbuilding structure, and the weight of an individual supported nonbuilding structure does not exceed the 25% limit but the combined weight of the supported nonbuilding structures does, it is recommended that the combined analysis and design approach of Section 15.3.2 be used. It is also suggested that dynamic analysis be performed in such cases, because the equivalent lateral force procedure may not capture some important response effects in some members of the supporting structure.

Where the weight of the supported nonbuilding structure does not exceed the 25% limit and a combined analysis is performed, the following procedure should be used to determine the F_p force of the supported nonbuilding structure based on Eq. 13.3-4:

1. A modal analysis should be performed in accordance with Section 12.9. The base shear of the combined structure and nonbuilding structure should be taken as no less than 85% of the equivalent lateral force procedure base shear.
2. For a component supported at level i, the acceleration at that level should be taken as a_i, the total shear just below level i divided by the seismic weight at and above level i.
3. The elastic value of the component shear force coefficient should next be determined as the shear force from the modal analysis at the point of attachment of the component to the structure divided by the weight of the component. This value is preliminarily taken as $a_i a_p$. Because a_p cannot be taken as less than 1.0, the value of a_p is taken as $a_i a_p / a_i$, except that the final value a_p need not be taken as greater than 2.5 and should not be taken as less than 1.0. The final value of $a_i a_p$ should be the final value of a_i determined in Step 2 multiplied by the final value of a_p determined earlier in this step.
4. The resulting value of $(a_i a_p)$ should be used in Eq. 13.3-4; the resulting value of F_p is subject to the maximum and minimum values of Eqs. 13.3-2 and 13.3-3, respectively.

C15.3.2 Greater Than or Equal to 25% Combined Weight Condition. Where the weight of the supported structure is relatively large compared with the weight of the supporting structure, the overall response can be affected significantly. The standard sets forth two analysis approaches, depending on the rigidity of the nonbuilding structure. The determination of what is deemed rigid or flexible is based on the same criteria used for nonstructural components.

Where the supported nonbuilding structure is rigid, it is acceptable to treat the supporting structure as a nonbuilding structure similar to a building and to determine its design loads and detailing using the requirements of Section 15.5. The design of the rigid nonbuilding structure and its anchorage is determined using the requirements of Chapter 13 with the amplification factor, a_p, taken as 1.0. However, this condition is relatively rare because the flexibility of any directly supporting members in the primary structure, such as floor beams, must be considered in determining the period of the component.

In the usual case, where the supported nonbuilding structure is flexible, a combined model of the supporting structure and the supported nonbuilding structure is used. The design loads and detailing are determined based on the lower R value of the supported nonbuilding structure or supporting structure.

Although not specifically mentioned in Section 15.3.2, another approach is permitted. A nonlinear response history analysis of the combined system can be performed in accordance with Section 16.2, and the results can be used for the design of both the supported and supporting nonbuilding structures. This option should be considered where standard static and dynamic elastic analysis approaches may be inadequate to evaluate the earthquake response (such as for suspended boilers). This option should be used with extreme caution because modeling and interpretation of results require considerable judgment. Because of this sensitivity, Section 16.2 requires independent design review.

C15.4 STRUCTURAL DESIGN REQUIREMENTS

This section specifies the basic coefficients and minimum design forces to be used to determine seismic design loads for nonbuilding structures. It also specifies height limits and restrictions. As with building structures, it presumes that the first step in establishing the design forces is to determine the design base shear for the structure.

There are two types of nonbuilding structures: those with structural systems similar to buildings and those with structural systems not similar to buildings. Specific requirements for these two cases appear in Sections 15.5 and 15.6.

Table 15.4-1 contains the response modification coefficient (R) for nonbuilding structures similar to buildings. Table 15.4-2 contains the response modification coefficient for nonbuilding structures not similar to buildings. Every response modification coefficient has associated design and detailing requirements to ensure the required ductility associated with that response modification coefficient value (e.g., AISC 341). Some structures, such as pipe racks, do not resemble a traditional building in that they do not house people or have such things as walls and bathrooms. These structures have lateral force-resisting systems composed of braced frames and moment frames similar to a traditional building. Therefore, pipe racks are considered nonbuilding structures similar to buildings. The response modification coefficient for a pipe rack should be taken from Table 15.4-1 for the appropriate lateral force-resisting system used, and the braced frames and/or moment frames used must meet all of the design and detailing requirements associated with the R value selected (see Section 15.5.2, Pipe Racks).

Most major power distribution facility (power island) structures, such as HRSG support structures, steam turbine pedestals, coal boiler support structures, pipe racks, air inlet structures, and duct support structures, also resist lateral forces predominantly by use of buildinglike framing systems such as moment frames, braced frames, or cantilever column systems. Therefore, their response modification coefficient should be selected from Table 15.4-1, and they must meet all the design and detailing requirements associated with the response modification coefficient selected.

Many nonbuilding structures, such as flat-bottom tanks, silos, and stacks, do not use braced frames or moment frames similar to those found in buildings to resist seismic loads. Therefore, they have their own unique response modification coefficient, which can be found in Table 15.4-2.

For nonbuilding structures with lateral systems composed predominantly of buildinglike framing systems, such as moment frames, braced frames, or cantilever column systems, it would

be inappropriate to extrapolate the descriptions in Table 15.4-2, resulting in inappropriately high response modification coefficients and the elimination of detailing requirements.

Once a response modification coefficient is selected from the tables, Section 15.4.1 provides additional guidance.

C15.4.1 Design Basis. Separate tables provided in this section identify the basic coefficients, associated detailing requirements, and height limits and restrictions for the two types of nonbuilding structures.

For nonbuilding structures similar to buildings, the design seismic loads are determined using the same procedures used for buildings as specified in Chapter 12, with two exceptions: fundamental periods are determined in accordance with Section 15.4.4, and Table 15.4-1 provides additional options for structural systems. Although only Section 12.8 (the equivalent lateral force procedure) is specifically mentioned in Section 15.4.1, Section 15.1.3 provides the analysis procedures that are permitted for nonbuilding structures.

In Table 15.4-1, seismic coefficients, system restrictions, and height limits are specified for a few nonbuilding structures similar to buildings. The values of R, Ω_0, and C_d; the detailing requirement references; and the structural system height limits are the same as those in Table 12.2-1 for the same systems, except for ordinary moment frames. In Chapter 12, increased height limits for ordinary moment frame structural systems apply to metal building systems, while in Chapter 15 they apply to pipe racks with end plate bolted moment connections. The seismic performance of pipe racks was judged to be similar to that of metal building structures with end plate bolted moment connections, so the height limits were made the same as those specified in previous editions.

Table 15.4-1 also provides lower R values with less restrictive height limits in SDCs D, E, and F based on good performance in past earthquakes. For some options, no seismic detailing is required if very low values of R (and corresponding high seismic design forces) are used. The concept of extending this approach to other structural systems is the subject of future research using the methodology developed by the ATC 63 project.

For nonbuilding structures not similar to buildings, the seismic design loads are determined as in Chapter 12 with three exceptions: the fundamental periods are determined in accordance with Section 15.4.4, the minima are those specified in Section 15.4.1.2, and the seismic coefficients are those specified in Table 15.4-2.

Some entries in Table 15.4-2 may seem to be conflicting or confusing. For example, the first major entry is for elevated tanks, vessels, bins, or hoppers. A subset of this entry is for tanks on braced or unbraced legs. This subentry is intended for structures where the supporting columns are integral with the shell (such as an elevated water tank). Tension-only bracing is allowed for such a structure. Where the tank or vessel is supported by buildinglike frames, the frames are to be designed in accordance with all of the restrictions normally applied to building frames. The entry for tanks or vessels supported on structural towers similar to buildings assumes that the operating weight of the supported tank or vessel is less than 25% of the total weight; if the ratio is greater than 25%, the proper entry is that most closely related to the subject vessel or bin.

C15.4.1.1 Importance Factor. The importance factor for a nonbuilding structure is based on the risk category defined in Chapter 1 of the standard or the building code being used in conjunction with the standard. In some cases, reference standards provide a higher importance factor, in which case the higher importance factor is used.

If the importance factor is taken as 1.0 based on a hazard and operability (HAZOP) analysis performed in accordance with Chapter 1, the third paragraph of Section 1.5.3 requires careful consideration; worst-case scenarios (instantaneous release of a vessel or piping system) must be considered. HAZOP risk analysis consultants often do not make such assumptions, so the design professional should review the HAZOP analysis with the HAZOP consultant to confirm that such assumptions have been made to validate adjustment of the importance factor. Clients may not be aware that HAZOP consultants do not normally consider the worst-case scenario of instantaneous release but tend to focus on other more hypothetical limited-release scenarios, such as those associated with a 2-inch square hole in a tank or vessel.

C15.4.2 Rigid Nonbuilding Structures. The definition of rigid (having a natural period of less than 0.06 s) was selected judgmentally. Below that period, the energy content of seismic ground motion is generally believed to be very low, and therefore the building response is not likely to be excessively amplified. Also, it is unlikely that any building will have a first mode period as low as 0.06 s, and it is even unusual for a second mode period to be that low. Thus, the likelihood of either resonant behavior or excessive amplification becomes quite small for equipment having periods below 0.06 s.

The analysis to determine the period of the nonbuilding structure should include the flexibility of the soil subgrade.

C15.4.3 Loads. As for buildings, the seismic weight must include the range of design operating weight of permanent equipment.

C15.4.4 Fundamental Period. A significant difference between building structures and nonbuilding structures is that the approximate period formulas and limits of Section 12.8.2.1 may not be used for nonbuilding structures. In lieu of calculating a specific period for a nonbuilding structure for determining seismic lateral forces, it is of course conservative to assume a period of $T = T_s$, which results in the largest lateral design forces. Computing the fundamental period is not considered a significant burden because most commonly used computer analysis programs can perform the required calculations.

C15.4.8 Site-Specific Response Spectra. Where site-specific response spectra are required, they should be developed in accordance with Chapter 21 of the standard. If determined for other recurrence intervals, Section 21.1 applies, but Sections 21.2 through 21.4 apply only to MCE determinations. Where other recurrence intervals are used, it should be demonstrated that the requirements of Chapter 15 also are satisfied.

C15.4.9 Anchors in Concrete or Masonry. Many nonbuilding structures rely on the ductile behavior of anchor bolts to justify the response modification factor, R, assigned to the structure. Nonbuilding structures typically rely more heavily on anchorage to provide system ductility. The additional requirements of Section 15.4.9 provide additional anchorage strength and ductility to support the response modification factors assigned to these systems. The addition of Section 15.4.9 provides a consistent treatment of anchorage for nonbuilding structures.

C15.5 NONBUILDING STRUCTURES SIMILAR TO BUILDINGS

C15.5.1 General. Although certain nonbuilding structures exhibit behavior similar to that of building structures, their functions and occupancies are different. Section 15.5 of the standard addresses the differences.

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C15.5.2 Pipe Racks. Free-standing pipe racks supported at or below grade with framing systems that are similar to building systems are designed in accordance with Section 12.8 or 12.9 and Section 15.4. Single-column pipe racks that resist lateral loads should be designed as inverted pendulums.

Based on good performance in past earthquakes, Table 15.4-1 sets forth the option of lower *R* values and less restrictive height limits for structural systems commonly used in pipe racks. The *R* value versus height limit trade-off recognizes that the size of some nonbuilding structures is determined by factors other than traditional loadings and results in structures that are much stronger than required for seismic loadings. Therefore, the ductility demand is generally much lower than that for a corresponding building. The intent is to obtain the same structural performance at the increased heights. This option proves to be economical in most situations because of the relative cost of materials and construction labor. The lower *R* values and increased height limits of Table 15.4-1 apply to nonbuilding structures similar to buildings; they cannot be applied to building structures. Table C15.5-1 illustrates the *R* values and height limits for a 70-ft-high steel ordinary moment frame (OMF) pipe rack.

C15.5.3 Steel Storage Racks. The two approaches to the design of steel storage racks set forth by the standard are intended to produce comparable results. The specific revisions to the RMI specification cited in earlier editions of this standard and the detailed requirements of the new ANSI/RMI 16.1 specification reflect the recommendations of FEMA 460, *Seismic Considerations for Steel Storage Racks Located in Areas Accessible to the Public.*

While the ANSI/RMI 16.1 specification reflects the recommendations of FEMA 460, the anchorage provisions of the ANSI/RMI 16.1 specification are not in conformance with ASCE/SEI 7. Therefore, specific anchorage requirements were added in Sections 15.5.3.2 and 15.5.3.3.

These recommendations address the concern that storage racks in warehouse-type retail stores may pose a greater seismic risk to the general public than exists in low-occupancy warehouses or more conventional retail environments. Under normal conditions, retail stores have a far higher occupant load than an ordinary warehouse of a comparable size. Failure of a storage rack system in a retail environment is much more likely to cause personal injury than would a similar failure in a storage warehouse. To provide an appropriate level of additional safety in areas open to the public, an importance factor of 1.50 is specified. Storage rack contents, though beyond the scope of the standard, may pose a potentially serious threat to life should they fall from the shelves in an earthquake. It is recommended that restraints be provided, as shown in Figure C15.5-1, to prevent the contents of rack shelving open to the general public from falling during strong ground shaking.

C15.5.4 Electrical Power Generating Facilities. Electrical power plants closely resemble building structures, and their performance in seismic events has been good. For reasons of mechanical performance, lateral drift of the structure must be limited. The lateral bracing system of choice has been the concentrically braced frame. In the past, the height limits on braced frames in particular have been an encumbrance to the design of large power generating facilities. Based on acceptable past performance, Table 15.4-1 permits the use of ordinary concentrically braced frames with both lower *R* values and less restrictive height limits. This option is particularly effective for boiler buildings that generally are 300 ft or more in height. A peculiarity of large boiler buildings is the general practice of suspending the boiler from the roof structures; this practice results in an unusual mass distribution, as discussed in Section C15.1.3.

C15.5.5 Structural Towers for Tanks and Vessels. The requirements of this section apply to structural towers that are not integral with the supported tank. Elevated water tanks designed in accordance with AWWA D100-05 are not subject to Section 15.5.5.

C15.5.6 Piers and Wharves. Current industry practice recognizes the distinct differences between the two categories of piers and wharves described in the standard. Piers and wharves with public occupancy, described in Section 15.5.6.2, are commonly treated as the "foundation" for buildings or buildinglike structures; design is performed using the standard, likely under the jurisdiction of the local building official. Piers and wharves without occupancy by the general public are often treated differently and are outside the scope of the standard; in many cases, these structures do not fall under the jurisdiction of building officials, and design is performed using other industry-accepted approaches.

Design decisions associated with these structures often reflect economic considerations by both owners and local, regional, or state jurisdictional entities with interest in commercial development. Where building officials have jurisdiction but lack experience analyzing pier and wharf structures, reliance on other industry-accepted design approaches is common.

Where occupancy by the general public is not a consideration, seismic design of structures at major ports and marine terminals often uses a performance-based approach, with criteria and methods that are very different from those used for buildings, as provided in the standard. Design approaches most commonly used are generally consistent with the practices and criteria described in the following documents:

1. *Seismic Design Guidelines for Port Structures.* (2001). Working Group No. 34 of the Maritime Navigation Commission (PIANC/MarCom/WG34), A. A. Balkema, Lisse, Netherlands.
2. Ferritto, J., Dickenson, S., Priestley, N., Werner, S., Taylor, C., Burke, D., Seelig, W., and Kelly, S. (1999). *Seismic Criteria for California Marine Oil Terminals*, Vol. 1 and Vol. 2, Technical Report TR-2103-SHR, Naval Facilities Engineering Service Center, Port Hueneme, Calif.
3. Priestley, N.J.N., Siebel, F., and Calvi, G. M. (1996). *Seismic Design and Retrofit of Bridges*, New York.

Table C15.5-1 *R* Value Selection Example for Steel OMF Pipe Racks

SDC	*R*	ASCE/SEI 7-10 Table	System	Seismic Detailing Requirements
C	3.5	12.2-1 or 15.4-1	Ordinary steel moment frame	AISC (2010a)
C	3	12.2-1	Structural steel systems not specifically detailed for seismic resistance	None
D or E	2.5	15.4-1	Steel OMF with permitted height increase	AISC (2010a)
D, E, or F	1	15.4-1	Steel OMF with unlimited height	AISC (2010b)

4. Werner, S. D., ed. (1998). *Seismic Guidelines for Ports*, Monograph No. 12, ASCE, Reston, Va.
5. *Marine Oil Terminal Engineering and Maintenance Standards.* (2005). Title 24, Part 2, California Building Code, Chapter 31F.

These alternative approaches have been developed over a period of many years by working groups within the industry, and they reflect the historical experience and performance characteristics of these structures, which are very different from those of building structures.

The main emphasis of the performance-based design approach is to provide criteria and methods that depend on the economic importance of a facility. Adherence to the performance criteria in the documents listed does not seek to provide uniform margins of collapse for all structures; their application is expected to provide at least as much inherent life-safety as for buildings designed using the standard. The reasons for the higher inherent level of life-safety for these structures include the following:

1. These structures have relatively infrequent occupancy, with few working personnel and very low density of personnel. Most of these structures consist primarily of open area, with no enclosed structures that can collapse onto personnel. Small control buildings on marine oil terminals or similar secondary structures are commonly designed in accordance with the local building code.
2. These pier or wharf structures typically are constructed of reinforced concrete, prestressed concrete, or steel and are highly redundant because of the large number of piles supporting a single wharf deck unit. Tests done at the University of California at San Diego for the Port of Los Angeles have shown that very high ductilities (10 or more) can be achieved in the design of these structures using practices currently used in California ports.
3. Container cranes, loading arms, and other major structures or equipment on piers or wharves are specifically designed not to collapse in an earthquake. Typically, additional piles and structural members are incorporated into the wharf or pier specifically to support such items.
4. Experience has shown that seismic "failure" of wharf structures in zones of strong seismicity is indicated not by collapse but by economically irreparable deformations of the piles. The wharf deck generally remains level or slightly tilting but shifts out of position. Earthquake loading on properly maintained marine structures has never induced complete failure that could endanger life-safety.
5. The performance-based criteria of the listed documents address reparability of the structure. These criteria are much more stringent than collapse prevention criteria and create a greater margin for life-safety.

Lateral load design of these structures in low, or even moderate, seismic regions often is governed by other marine conditions.

C15.6 GENERAL REQUIREMENTS FOR NONBUILDING STRUCTURES NOT SIMILAR TO BUILDINGS

Nonbuilding structures not similar to buildings exhibit behavior markedly different from that of building structures. Most of these types of structures have reference documents that address their unique structural performance and behavior. The ground motion in the standard requires appropriate translation to allow use with industry standards.

C15.6.1 Earth-Retaining Structures. Section C11.8.3 presents commonly used approaches for the design of nonyielding walls and yielding walls for bending, overturning, and sliding, taking into account the varying soil types, importance, and site seismicity.

C15.6.2 Stacks and Chimneys. The design of stacks and chimneys to resist natural hazards generally is governed by wind design considerations. The exceptions to this general rule involve locations with high seismicity, stacks and chimneys with large elevated masses, and stacks and chimneys with unusual geometries. It is prudent to evaluate the effect of seismic loads in all but those areas with the lowest seismicity. Although not specifically required, it is recommended that the special seismic details required elsewhere in the standard be considered for application to stacks and chimneys.

Concrete chimneys typically possess low ductility, and their performance is especially critical in the regions around large (breach) openings because of reductions in strength and loss of confinement for vertical reinforcement in the jamb regions around the openings. Earthquake-induced chimney failures have occurred in recent history (in Turkey in 1999) and have been attributed to strength and detailing problems (Kilic and Sozen 2003). Therefore, the R value of 3 traditionally used in ASCE/SEI 7-05 for concrete stacks and chimneys is reduced to 2, and detailing requirements for breach openings are added in the 2010 edition of this standard.

Guyed steel stacks and chimneys generally are lightweight. As a result, the design loads caused by natural hazards generally are governed by wind. On occasion, large flares or other elevated masses located near the top may require in-depth seismic analysis. Although it does not specifically address seismic loading, Chapter 6 of Troitsky (1990) provides a methodology appropriate for resolution of the seismic forces defined in the standard.

C15.6.4 Special Hydraulic Structures. The most common special hydraulic structures are baffle walls and weirs that are used in water-treatment and wastewater-treatment plants. Because there are openings in the walls, during normal operations the fluid levels are equal on each side of the wall, exerting no net horizontal force. Sloshing during a seismic event can exert large forces on the wall, as illustrated in Figure C15.6-1. The walls can fail unless they are designed properly to resist the dynamic fluid forces.

C15.6.5 Secondary Containment Systems. This section reflects the judgment that designing all impoundment dikes for the MCE ground motion when full and sizing all impoundment dikes for the sloshing wave is too conservative. Designing an impoundment dike as full for the MCE assumes failure of the primary containment and occurrence of a significant aftershock. Such significant aftershocks (of the same magnitude as the MCE ground motion) are rare and do not occur in all locations. Although explicit design for aftershocks is not a requirement of the standard, secondary containment must be designed full for an aftershock to protect the general public. The use of two-thirds of the MCE ground motion as the magnitude of the design aftershock is supported by Bath's law, according to which the maximum expected aftershock magnitude may be estimated to be 1.2 scale units below the main shock magnitude.

The risk assessment and risk management plan described in Section 1.5.2 are used to determine where the secondary containment must be designed full for the MCE. The decision to design secondary containment for this more severe condition should be based on the likelihood of a significant aftershock occurring at the particular site, considering the risk posed to the general

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public by the release of hazardous material from the secondary containment.

Secondary containment systems must be designed to contain the sloshing wave where the release of liquid would place the general public at risk by exposing them to hazardous materials, by scouring of foundations of adjacent structures, or by causing other damage to adjacent structures.

C15.6.5.1 Freeboard. Equation 15.6-1 was revised to return to the more exact theoretical formulation for sloshing wave height instead of the rounded value introduced in ASCE/SEI 7-05. The rounded value in part accounted for maximum direction of response effects. Because the ground motion definition in ASCE/SEI 7-10 is changed and the maximum direction of response is now directly accounted for, it is no longer necessary to account for these effects by rounding up the theoretical sloshing wave height factor in Eq. 15.6-1.

C15.6.6 Telecommunication Towers. Telecommunication towers support small masses, and their design generally is governed by wind forces. Although telecommunication towers have a history of experiencing seismic events without failure or significant damage, seismic design in accordance with the standard is required.

Typically bracing elements bolt directly (without gusset plates) to the tower legs, which consist of pipes or bent plates in a triangular plan configuration.

C15.7 TANKS AND VESSELS

C15.7.1 General. Methods for seismic design of tanks, currently adopted by a number of reference documents, have evolved from earlier analytical work by Jacobsen, Housner, Veletsos, Haroun, and others. The procedures used to design flat-bottom storage tanks and liquid containers are based on the work of Housner, Wozniak, and Mitchell. The reference documents for tanks and vessels have specific requirements to safeguard against catastrophic failure of the primary structure based on observed behavior in seismic events since the 1930s. Other methods of analysis, using flexible shell models, have been proposed but at present are beyond the scope of the standard.

The industry-accepted design methods use three basic steps:

1. Dynamic modeling of the structure and its contents. When a liquid-filled tank is subjected to ground acceleration, the lower portion of the contained liquid, identified as the impulsive component of mass, W_i, acts as if it were a solid mass rigidly attached to the tank wall. As this mass accelerates, it exerts a horizontal force, P_i, on the wall; this force is directly proportional to the maximum acceleration of the tank base. This force is superimposed on the inertia force of the accelerating wall itself, P_s. Under the influence of the same ground acceleration, the upper portion of the contained liquid responds as if it were a solid mass flexibly attached to the tank wall. This portion, which oscillates at its own natural frequency, is identified as the convective component, W_c, and exerts a horizontal force, P_c, on the wall. The convective component oscillations are characterized by sloshing whereby the liquid surface rises above the static level on one side of the tank and drops below that level on the other side.
2. Determination of the period of vibration, T_i, of the tank structure and the impulsive component and determination of the natural period of oscillation (sloshing), T_c, of the convective component.
3. Selection of the design response spectrum. The response spectrum may be site specific, or it may be constructed on the basis of seismic coefficients given in national codes and standards. Once the design response spectrum is constructed, the spectral accelerations corresponding to T_i and T_c are obtained and are used to calculate the dynamic forces P_i, P_s, and P_c.

Detailed guidelines for the seismic design of circular tanks, incorporating these concepts to varying degrees, have been the province of at least four industry reference documents: AWWA D100 for welded steel tanks (since 1964); API 650 for petroleum storage tanks; AWWA D110 for prestressed, wire-wrapped tanks (since 1986); and AWWA D115 for prestressed concrete tanks stressed with tendons (since 1995). In addition, API 650 and API 620 contain provisions for petroleum, petrochemical, and cryogenic storage tanks. The detail and rigor of analysis prescribed in these documents have evolved from a semistatic approach in the early editions to a more rigorous approach at present, reflecting the need to include the dynamic properties of these structures.

The requirements in Section 15.7 are intended to link the latest procedures for determining design-level seismic loads with the allowable stress design procedures based on the methods in the standard. These requirements, which in many cases identify specific substitutions to be made in the design equations of the reference documents, will assist users of the standard in making consistent interpretations.

ACI has published ACI 350.3-06, "Seismic Design of Liquid-Containing Concrete Structures." This document, which addresses all types of concrete tanks (prestressed and nonprestressed, circular, and rectilinear), has provisions that are unfortunately not consistent with the seismic criteria of ASCE/SEI 7-10. However, the document, when combined with the modifications required in Section 15.7.7.3, serves as both a practical "how-to" loading reference and a guide to supplement application of ACI 318 Chapter 21.

C15.7.2 Design Basis. In the case of the seismic design of nonbuilding structures, standardization requires adjustments to industry reference documents to minimize existing inconsistencies among them, while recognizing that structures designed and built over the years in accordance with these documents have performed well in earthquakes of varying severity. Of the inconsistencies among reference documents, the ones most important to seismic design relate to the base shear equation. The traditional base shear takes the following form:

$$V = \frac{ZIS}{R_w} CW \qquad \text{(C15.7-1)}$$

An examination of those terms as used in the different references reveals the following:

1. *Z, S:* The seismic zone coefficient, Z, has been rather consistent among all the documents because it usually has been obtained from the seismic zone designations and maps in the model building codes. However, the soil profile coefficient, S, does vary from one document to another. In some documents, these two terms are combined.
2. *I*: The importance factor, I, has varied from one document to another, but this variation is unavoidable and understandable because of the multitude of uses and degrees of importance of tanks and vessels.
3. *C:* The coefficient C represents the dynamic amplification factor that defines the shape of the design response

spectrum for any given ground acceleration. Because C is primarily a function of the frequency of vibration, inconsistencies in its derivation from one document to another stem from at least two sources: differences in the equations for the determination of the natural frequency of vibration, and differences in the equation for the coefficient itself. (For example, for the shell/impulsive liquid component of lateral force, the steel tank documents use a constant design spectral acceleration [constant C] that is independent of the "impulsive" period, T.) In addition, the value of C varies depending on the damping ratio assumed for the vibrating structure (usually between 2% and 7% of critical).

4. Where a site-specific response spectrum is available, calculation of the coefficient C is not necessary except in the case of the convective component (coefficient C_c), which is assumed to oscillate with 0.5% of critical damping and whose period of oscillation is usually long (greater than 2.5 s). Because site-specific spectra are usually constructed for high damping values (3% to 7% of critical) and because the site-specific spectral profile may not be well defined in the long-period range, an equation for C_c applicable to a 0.5% damping ratio is necessary to calculate the convective component of the seismic force.
5. R_w: The response modification factor, R_w, is perhaps the most difficult to quantify, for a number of reasons. Although R_w is a compound coefficient that is supposed to reflect the ductility, energy-dissipating capacity, and redundancy of the structure, it is also influenced by serviceability considerations, particularly in the case of liquid-containing structures.

In the standard, the base shear equation for most structures has been reduced to $V = C_sW$, where the seismic response coefficient, C_s, replaces the product ZSC/R_w. C_s is determined from the design spectral response acceleration parameters S_{DS} and S_{D1} (at short periods and at a period of 1, respectively), which in turn are obtained from the mapped MCE spectral accelerations S_s and S_1. As in the case of the prevailing industry reference documents, where a site-specific response spectrum is available, C_s is replaced by the actual values of that spectrum.

The standard contains several bridging equations, each designed to allow proper application of the design criteria of a particular reference document in the context of the standard. These bridging equations associated with particular types of liquid-containing structures and the corresponding reference documents are discussed in the following. Calculation of the periods of vibration of the impulsive and convective components is in accordance with the reference documents, and the detailed resistance and allowable stresses for structural elements of each industry structure are unchanged, except where new information has led to additional requirements.

It is expected that the bridging equations of Sections 15.7.7.3 and 15.7.10.7 will be eliminated as the relevant reference documents are updated to conform to the standard. The bridging equations previously provided for AWWA D100 and API 650 already have been eliminated as a result of updates of these documents.

C15.7.3 Strength and Ductility. As is the case for building structures, ductility and redundancy in the lateral support systems for tanks and vessels are desirable and necessary for good seismic performance. Tanks and vessels are not highly redundant structural systems, and therefore ductile materials and well designed connection details are needed to increase the capacity of the vessel to absorb more energy without failure. The critical performance of many tanks and vessels is governed by shell stability requirements rather than by yielding of the structural elements. For example, contrary to building structures, ductile stretching of anchor bolts is a desirable energy absorption component where tanks and vessels are anchored. The performance of cross-braced towers is highly dependent on the ability of the horizontal compression struts and connection details to develop fully the tension yielding in the rods. In such cases, it is also important to preclude both premature failure in the threaded portion of the connection and failure of the connection of the rod to the column before yielding of the rod.

The changes made to Section 15.7.3(a) are intended to ensure that anchors and anchor attachments are designed such that the anchor yields (stretches) before the anchor attachment to the structure fails. The changes also clarify that the anchor rod embedment requirements are to be based on the requirements of Section 15.7.5 and not Section 15.7.3(a).

C15.7.4 Flexibility of Piping Attachments. Poor performance of piping connections (tank leakage and damage) caused by seismic deformations is a primary weakness observed in seismic events. Although commonly used piping connections can impart mechanical loads to the tank shell, proper design in seismic areas results in only negligible mechanical loads on tank connections subject to the displacements shown in Table 15.7-1. API 650 treats the values shown in Table 15.7-1 as allowable stress-based values and therefore requires that these values be multiplied by 1.4 where strength-based capacity values are required for design.

The displacements shown in Table 15.7-1 are based on movements observed during past seismic events. The vertical tank movements listed are caused by stretch of the mechanical anchors or steel tendons (in the case of a concrete tank) for mechanically anchored tanks or the deflection caused by bending of the bottom of self-anchored tanks. The horizontal movements listed are caused by the deformation of the tank at the base.

In addition, interconnected equipment, walkways, and bridging between multiple tanks must be designed to resist the loads and accommodate the displacements imposed by seismic forces. Unless connected tanks and vessels are founded on a common rigid foundation, the calculated differential movements must be assumed to be out of phase.

C15.7.5 Anchorage. Many steel tanks can be designed without anchors by using annular plate detailing in accordance with reference documents. Where tanks must be anchored because of overturning potential, proper anchorage design provides both a shell attachment and an embedment detail that will allow the bolt to yield without tearing the shell or pulling the bolt out of the foundation. Properly designed anchored tanks have greater reserve strength to resist seismic overload than do unanchored tanks.

To ensure that the bolt yields (stretches) before failure of the anchor embedment, the anchor embedment must be designed in accordance with ACI 318, Appendix D, Equation D-3 and must be provided with a minimum gauge length of eight bolt diameters. Gauge length is the length of the bolt that is allowed to stretch. It may include part of the embedment length into the concrete that is not bonded to the bolt. A representation of gauge length is shown in Figure C15.7-1.

It is also important that the bolt not be significantly oversized to ensure that the bolt stretches. The prohibition on using the load combinations with overstrength of Section 12.4.3 is intended to accomplish this goal.

Where anchor bolts and attachments are misaligned such that the anchor nut or washer does not bear evenly on the attachment,

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additional bending stresses in threaded areas may cause premature failure before anchor yielding.

C15.7.6 Ground-Supported Storage Tanks for Liquids

C15.7.6.1 General. The response of ground storage tanks to earthquakes is well documented by Housner, Mitchell and Wozniak, Veletsos, and others. Unlike building structures, the structural response of these tanks is influenced strongly by the fluid-structure interaction. Fluid-structure interaction forces are categorized as sloshing (convective) and rigid (impulsive) forces. The proportion of these forces depends on the geometry (height-to-diameter ratio) of the tank. API 650, API 620, AWWA D100, AWWA D110, AWWA D115, and ACI 350.3 provide the data necessary to determine the relative masses and moments for each of these contributions.

The standard requires that these structures be designed in accordance with the prevailing reference documents, except that the height of the sloshing wave, δ_s, must be calculated using Eq. 15.7-13. API 650 and AWWA D100 include this requirement in their latest editions.

Equations 15.7-10 and 15.7-11 provide the spectral acceleration of the sloshing liquid for the constant-velocity and constant-displacement regions of the response spectrum, respectively. The 1.5 factor in these equations is an adjustment for 0.5% damping. An exception in the use of Eq. 15.7-11 was added for the 2010 edition of this standard. The mapped values of T_L were judged to be unnecessarily conservative by the ASCE 7 Seismic Subcommittee in light of actual site-specific studies carried out since the introduction of the T_L requirements of ASCE/SEI 7-05. These studies indicate that the mapped values of T_L appear to be very conservative based on observations during recent large earthquakes, especially the 2010 M_w 8.8 Chilean earthquake, where the large amplifications at very long periods (6-10 s) were not evident either in the ground motion records or in the behavior of long-period structures (particularly sloshing in tanks). Because a revision of the T_L maps is a time-consuming task that would not be possible during the 2010 update cycle, an exception was added to allow the use of site-specific values that are less than the mapped values with a floor of 4 s or one-half the mapped value of T_L. The exception was added under Section 15.7.6 because, for nonbuilding structures, the overly conservative values for T_L are primarily an issue for tanks and vessels. Discussion of the site-specific procedures can be found in the Commentary for Chapter 22.

Small-diameter tanks and vessels are more susceptible to overturning and vertical buckling. As a general rule, a greater ratio of *H*/*D* produces lower resistance to vertical buckling. Where *H*/*D* is greater than 2, overturning approaches "rigid mass" behavior (the sloshing mass is small). Large-diameter tanks may be governed by additional hydrodynamic hoop stresses in the middle regions of the shell.

The impulsive period (the natural period of the tank components and the impulsive component of the liquid) is typically in the 0.25-0.6 s range. Many methods are available for calculating the impulsive period. The Veletsos flexible-shell method is commonly used by many tank designers. For example, see Velestos (1974) and Malhotra et al. (2000).

C15.7.6.1.1 Distribution of Hydrodynamic and Inertia Forces. Most of the reference documents for tanks define reaction loads at the base of shell-foundation interface, without indicating the distribution of loads on the shell as a function of height. ACI 350.3 specifies the vertical and horizontal distribution of such loads.

The overturning moment at the base of the shell in the industry reference documents is only the portion of the moment that is transferred to the shell. The total overturning moment also includes the variation in bottom pressure, which is an important consideration for design of pile caps, slabs, or other support elements that must resist the total overturning moment. Wozniak and Mitchell (1978) and U.S. Department of Energy (1963, TID-7024) provide additional information.

C15.7.6.1.2 Sloshing. In past earthquakes, sloshing contents in ground storage tanks have caused both leakage and noncatastrophic damage to the roof and internal components. Even this limited damage and the associated costs and inconvenience can be significantly mitigated where the following items are considered:

1. Effective masses and hydrodynamic forces in the container;
2. Impulsive and pressure loads at
   a. The sloshing zone (that is, the upper shell and edge of the roof system);
   b. The internal supports (such as roof support columns and tray supports); and
   c. The internal equipment (such as distribution rings, access tubes, pump wells, and risers); and
3. Freeboard (which depends on the sloshing wave height).

When no freeboard is required, a minimum freeboard of 0.7 δs is recommended for economic considerations. Freeboard is always required for tanks assigned to Risk Category IV.

Tanks and vessels storing biologically or environmentally benign materials typically do not require freeboard to protect the public health and safety. However, providing freeboard in areas of frequent seismic occurrence for vessels normally operated at or near top capacity may lessen damage (and the cost of subsequent repairs) to the roof and upper container.

The sloshing wave height specified in Section 15.7.6.1.2 is based on the design earthquake defined in the standard. For economic reasons, freeboard for tanks assigned to Risk Category I, II, or III may be calculated using a fixed value of T_L equal to 4 s (as indicated in Section 15.7.6.1, note 4) but using the appropriate importance factor taken from Table 1.5-2. Because of life-safety concerns, freeboard for tanks assigned to Risk Category IV must be based on the mapped value of T_L. Because use of the mapped value of T_L results in the theoretical maximum value of freeboard, the calculation of freeboard in the case of Risk Category IV tanks is based on an importance factor equal to 1.0 (as indicated in Section 15.7.6.1, note 3).

If the freeboard provided is less than the computed sloshing height, δ_s, the sloshing liquid impinges on the roof in the vicinity of the roof-to-wall joint, subjecting it to a hydrodynamic force. This force may be approximated by considering the sloshing wave as a hypothetical static liquid column having a height, δ_s. The pressure exerted at any point along the roof at a distance y_s above the at-rest surface of the stored liquid may be assumed equal to the hydrostatic pressure exerted by the hypothetical liquid column at a distance $\delta_s - y_s$ from the top of that column. A better approximation of the pressure exerted on the roof is found in Malhotra (2005 and 2006).

Another effect of a less-than-full freeboard is that the restricted convective (sloshing) mass "converts" into an impulsive mass, thus increasing the impulsive forces. This effect should be taken into account in the tank design. A method for converting the restricted convective mass into an impulsive mass is found in Malhotra (2005 and 2006). It is recommended that sufficient

freeboard to accommodate the full sloshing height be provided wherever possible.

Equation 15.7-13 was revised to use the theoretical formulation for sloshing wave height instead of the rounded value introduced in ASCE/SEI 7-05. The rounded value of Eq. 15.6-1 increased the required freeboard by approximately 19%, thereby significantly increasing the cost of both secondary containment and large-diameter, ground-supported storage tanks. See Section C15.6.5.1 for additional commentary on freeboard.

C15.7.6.1.4 Internal Elements. Wozniak and Mitchell (1978) provide a recognized analysis method for determining the lateral loads on internal components caused by sloshing liquid.

C15.7.6.1.5 Sliding Resistance. Historically, steel ground-supported tanks full of product have not slid off foundations. A few unanchored, empty tanks or bulk storage tanks without steel bottoms have moved laterally during earthquake ground shaking. In most cases, these tanks may be returned to their proper locations. Resistance to sliding is obtained from the frictional resistance between the steel bottom and the sand cushion on which bottoms are placed. Because tank bottoms usually are crowned upward toward the tank center and are constructed of overlapping, fillet-welded, individual steel plates (resulting in a rough bottom), it is reasonably conservative to take the ultimate coefficient of friction on concrete as 0.70 (AISC 1986), and therefore a value of tan 30° (= 0.577) for sand is used in design. The value of 30° represents the internal angle of friction of sand and is conservatively used in design. The vertical weight of the tank and contents, as reduced by the component of vertical acceleration, provides the net vertical load. An orthogonal combination of vertical and horizontal seismic forces, following the procedure in Section 12.5.3, may be used. In recent years, a significant issue has been the prevention of subsurface pollution caused by tank bottom corrosion and leakage. To prevent this problem, liners are often used with the tank foundation. When some of these liners are used, sliding of the tank and/or foundation caused by the seismic base shear may be an issue. If the liner is completely contained within a concrete ring-wall foundation, the liner's surface is not the critical plane to check for sliding. If the liner is placed within an earthen foundation or is placed above or completely below a concrete foundation, it is imperative that sliding be evaluated. It is recommended that the sliding resistance factor of safety be at least 1.5.

C15.7.6.1.6 Local Shear Transfer. The transfer of seismic shear from the roof to the shell and from the shell to the base is accomplished by a combination of membrane shear and radial shear in the wall of the tank. For steel tanks, the radial (out-of-plane) seismic shear is very small and usually is neglected; thus, the shear is assumed to be resisted totally by membrane (in-plane) shear. For concrete walls and shells, which have a greater radial shear stiffness, the shear transfer may be shared. The ACI 350.3 commentary provides further discussion.

C15.7.6.1.7 Pressure Stability. Internal pressure may increase the critical buckling capacity of a shell. Provision to include pressure stability in determining the buckling resistance of the shell for overturning loads is included in AWWA D100. Recent testing on conical and cylindrical shells with internal pressure yielded a design methodology for resisting permanent loads in addition to temporary wind and seismic loads (Miller et al. 1997).

C15.7.6.1.8 Shell Support. Anchored steel tanks should be shimmed and grouted to provide proper support for the shell and to reduce impact on the anchor bolts under reversible loads. The high bearing pressures on the toe of the tank shell may cause inelastic deformations in compressible material (such as fiberboard), creating a gap between the anchor and the attachment. As the load reverses, the bolt is no longer snug and an impact of the attachment on the anchor can occur. Grout is a structural element and should be installed and inspected as an important part of the vertical and lateral force-resisting system.

C15.7.6.1.9 Repair, Alteration, or Reconstruction. During their service life, storage tanks are frequently repaired, modified, or relocated. Repairs often are related to corrosion, improper operation, or overload from wind or seismic events. Modifications are made for changes in service, updates to safety equipment for changing regulations, or installation of additional process piping connections. It is imperative that these repairs and modifications be designed and implemented properly to maintain the structural integrity of the tank or vessel for seismic loads and the design operating loads.

The petroleum steel tank industry has developed specific guidelines in API 653 that are statutory requirements in some states. It is recommended that the provisions of API 653 also be applied to other liquid storage tanks (e.g., water, wastewater, and chemical) as it relates to repairs, modifications, or relocation that affect the pressure boundary or lateral force-resisting system of the tank or vessel.

C15.7.7 Water Storage and Water Treatment Tanks and Vessels. The AWWA design requirements for ground-supported steel water storage structures use allowable stress design procedures that conform to the requirements of the standard.

C15.7.7.3 Reinforced and Prestressed Concrete. A review of ACI 350.3-06, Seismic Design of Liquid-Containing Concrete Structures and Commentary, revealed that this document is not in general agreement with the seismic provisions of ASCE/SEI 7.

This section was clarified to note that the importance factor, I, and the response modification factor, R, are to be specified by ASCE/SEI 7 and not the reference document. The descriptions used in ACI 350.3 to determine the applicable values of the importance factor and response modification factor do not match those used in ASCE/SEI 7.

It was noted that the ground motions for determining the convective (sloshing) seismic forces specified in ACI 350.3 were not the same and are actually lower than those specified by ASCE/SEI 7. ACI 350.3 essentially redefines the long-period transition period, T_L. This alternate transition period allows large-diameter tanks to have significantly lower convective forces and lower seismic freeboard than those permitted by the provisions of ASCE/SEI 7. Therefore, Section 15.7.7.3 was revised to require that the convective acceleration be determined according to the procedure found in Section 15.7.6.1.

It was also noted that the vertical ground motions specified in ACI 350.3 were not the same as those specified by ASCE/SEI 7. Therefore, appropriate modifications to ACI 350.3 Section 9.4.3 were introduced.

C15.7.8 Petrochemical and Industrial Tanks and Vessels Storing Liquids

C15.7.8.1 Welded Steel. The American Petroleum Institute (API) uses an allowable stress design procedure that conforms to the requirements of the standard.

The most common damage to tanks observed during past earthquakes includes the following:

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1. Buckling of the tank shell near the base because of excessive axial membrane forces. This buckling damage is usually evident as “elephant foot” buckles a short distance above the base or as diamond-shaped buckles in the lower ring. Buckling of the upper ring also has been observed.
2. Damage to the roof caused by impingement on the underside of the roof of sloshing liquid with insufficient freeboard.
3. Failure of piping or other attachments that are overly restrained.
4. Foundation failures.

Other than the above damage, the seismic performance of floating roofs during earthquakes has generally been good, with damage usually confined to the rim seals, gauge poles, and ladders. However, floating roofs have sunk in some earthquakes because of lack of adequate freeboard or the proper buoyancy and strength required by API 650. Similarly, the performance of open-top tanks with top wind girder stiffeners designed per API 650 has been generally good.

C15.7.8.2 Bolted Steel. Bolted steel tanks are often used for temporary functions. Where use is temporary, it may be acceptable to the jurisdictional authority to design bolted steel tanks for no seismic loads or for reduced seismic loads based on a reduced return period. For such reduced loads based on reduced exposure time, the owner should include a signed removal contract with the fixed removal date as part of the submittal to the authority having jurisdiction.

C15.7.9 Ground-Supported Storage Tanks for Granular Materials

C15.7.9.1 General. The response of a ground-supported storage tank storing granular materials to a seismic event is highly dependent on its height-to-diameter (*H*/*D*) ratio and the characteristics of the stored product. The effects of intergranular friction are described in more detail in C15.7.9.3.1 (increased lateral pressure), C15.7.9.3.2 (effective mass), and C15.7.9.3.3 (effective density).

Long-term increases in shell hoop tension because of temperature changes after the product has been compacted also must be included in the analysis of the shell; Anderson (1966) provides a suitable method.

C15.7.9.2 Lateral Force Determination. Seismic forces acting on ground-supported liquid storage tanks are divided between impulsive and convective (sloshing) components. However, in a ground-supported storage tank for granular materials, all seismic forces are of the impulsive type and relate to the period of the storage tank itself. Because of the relatively short period of a tank shell, the response is normally in the constant acceleration region of the response spectrum, which relates to S_{DS}. Therefore, the seismic base shear is calculated as follows:

$$V = \frac{S_{DS}}{\left(\frac{R}{I}\right)} W_{Effective} \qquad \text{(C15.7-2)}$$

where V, S_{DS}, I, and R have been previously defined, and $W_{\text{effective}}$ is the gross weight of the stored product multiplied by an effective mass factor and an effective density factor, as described in Sections C15.7.9.3.2 and C15.7.9.3.3, plus the dead weight of the tank. Unless substantiated by testing, it is recommended that the product of the effective mass factor and the effective density factor be taken as no less than 0.5 because of the limited test data and the highly variable properties of the stored product.

C15.7.9.3 Force Distribution to Shell and Foundation

C15.7.9.3.1 Increased Lateral Pressure. In a ground-supported tank storing granular materials, increased lateral pressures develop as a result of rigid body forces that are proportional to ground acceleration. Information concerning design for such pressure is scarce. Trahair et al. (1983) describes both a very simple, conservative method and a very difficult, analytical method using failure wedges based on the Mononobe–Okabe modifications of the classical Coulomb method.

C15.7.9.3.2 Effective Mass. For ground-supported tanks storing granular materials, much of the lateral seismic load can be transferred directly into the foundation, via intergranular shear, before it can reach the tank shell. The effective mass that loads the tank shell is highly dependent on the *H*/*D* ratio of the tank and the characteristics of the stored product. Quantitative information concerning this effect is scarce, but Trahair et al. (1983) describe a simple, conservative method to determine the effective mass. That method presents reductions in effective mass, which may be significant, for *H*/*D* ratios less than 2. This effect is absent for elevated tanks.

C15.7.9.3.3 Effective Density. Granular material stored in tanks (both ground-supported and elevated) does not behave as a solid mass. Energy loss through intergranular movement and grain-to-grain friction in the stored material effectively reduces the mass subject to horizontal acceleration. This effect may be quantified by an effective density factor less than 1.0.

Based on Chandrasekaran and Jain (1968) and on shake-table tests reported in Chandrasekaran et al. (1968), ACI 313 (1997) recommends an effective density factor of not less than 0.8 for most granular materials. According to Chandrasekaran and Jain (1968), an effective density factor of 0.9 is more appropriate for materials with high moduli of elasticity, such as aggregates and metal ores.

C15.7.9.3.4 Lateral Sliding. Most ground-supported steel storage tanks for granular materials rest on a base ring and do not have a steel bottom. To resist seismic base shear, a partial bottom or annular plate is used in combination with anchor bolts or a curb angle. An annular plate can be used alone to resist the seismic base shear through friction between the plate and the foundation, in which case the friction limits of Section 15.7.6.1.5 apply. The curb angle detail serves to keep the base of the shell round while allowing it to move and flex under seismic load. Various base details are shown in Figure 13 of Kaups and Lieb (1985).

C15.7.9.3.5 Combined Anchorage Systems. This section is intended to apply to combined anchorage systems that share loads based on their relative stiffnesses, and not to systems where sliding is resisted completely by one system (such as a steel annular plate) and overturning is resisted completely by another system (such as anchor bolts).

C15.7.10 Elevated Tanks and Vessels for Liquids and Granular Materials

C15.7.10.1 General. The three basic lateral load-resisting systems for elevated water tanks are defined by their support structure:

1. Multileg braced steel tanks (trussed towers, as shown in Fig. C15.7-2);
2. Small-diameter single-pedestal steel tanks (cantilever columns, as shown in Fig. C15.7-3); and

3. Large-diameter single-pedestal tanks of steel or concrete construction (load-bearing shear walls, as shown in Fig. C15.7-4).

Unbraced multileg tanks are uncommon. These types of tanks differ in their behavior, redundancy, and resistance to overload. Multileg and small-diameter pedestal tanks have longer fundamental periods (typically greater than 2 s) than the shear wall type tanks (typically less than 2 s). The lateral load failure mechanisms usually are brace failure for multileg tanks, compression buckling for small-diameter steel tanks, compression or shear buckling for large-diameter steel tanks, and shear failure for large-diameter concrete tanks. Connection, welding, and reinforcement details require careful attention to mobilize the full strength of these structures. To provide a greater margin of safety, R factors used with elevated tanks typically are less than those for other comparable lateral load-resisting systems.

C15.7.10.4 Transfer of Lateral Forces into Support Tower. The vertical loads and shears transferred at the base of a tank or vessel supported by grillage or beams typically vary around the base because of the relative stiffness of the supports, settlements, and variations in construction. Such variations must be considered in the design for vertical and horizontal loads.

C15.7.10.5 Evaluation of Structures Sensitive to Buckling Failure. Nonbuilding structures that are designed with limited structural redundancy for lateral loads may be susceptible to total failure when loaded beyond the design loads. This is particularly true for shell-type structures that exhibit unstable postbuckling behavior, such as tanks and vessels supported on shell skirts or pedestals. Evaluation for this critical condition ensures stability of the nonbuilding structure for governing design loads.

The design spectral response acceleration, S_a, used in this evaluation includes site factors. The I/R coefficient is taken as 1.0 for this critical check. The structural capacity of the shell is taken as the critical buckling strength (that is, the factor of safety is 1.0). Vertical and orthogonal combinations need not be considered for this evaluation because the probability of peak values occurring simultaneously is very low.

The intent of Section 15.7.10.5 and Table 15.4-2 is that skirt-supported vessels must be checked for seismic loads based on $I_{e/R}$ = 1.0 if the structure falls in Risk Category IV or if an R value of 3.0 is used in the design of the vessel. For the purposes of this section, a skirt is a thin-walled steel cylinder or cone used to support the vessel in compression. Skirt-supported vessels fail in buckling, which is not a ductile failure mode. Therefore, a more conservative design approach is required. The $I_{e/R}$ = 1.0 check typically governs the design of the skirt over using loads determined with an R factor of 3 in a moderate to high area of seismic activity. The only benefit of using an R factor of 3 in this case is in the design of the foundation. The foundation is not required to be designed for the $I_{e/R}$ = 1.0 load. Section 15.7.10.5, item b, states that resistance of the structure shall be defined as the critical buckling resistance of the element for the $I_{e/R}$ = 1.0 load. This stipulation means that the support skirt can be designed based on critical buckling (factor of safety of 1.0). The critical buckling strength of a skirt can be determined using a number of published sources. The two most common methods for determining the critical buckling strength of a skirt are the 2007 ASME BVPC, Section VIII, Division 2, 2008 Addenda, Paragraph 4.4, using a factor of safety of 1.0 and AWWA D100-05 Section 13.4.3.4. To use these methods, the radius, length, and thickness of the skirt, modulus of elasticity of the steel, and yield strength of the steel are required. These methods take into account both local buckling and slenderness effects of the skirt. Under no circumstance should the theoretical buckling strength of a cylinder, found in many engineering mechanics texts, be used to determine the critical buckling strength of the skirt. The theoretical value, based on a perfect cylinder, does not take into account imperfections built into real skirts. The theoretical buckling value is several times greater than the actual value measured in tests. The buckling values found in the suggested references above are based on actual tests.

Examples of applying the 2007 ASME BVPC, Section VIII, Division 2, 2008 Addenda, Paragraph 4.4 and AWWA D100-05 Section 13.4.3.4 buckling rules are shown in Fig. Ex-1.

Example Problem 1. 2007 ASME BPVC, Section VIII, Division 2, 2008 Addenda, paragraph 4.4

Vessel Period

$$T = \frac{7.78}{10^6}\left(\frac{H}{D}\right)^2 \sqrt{\frac{12wD}{t}} = \frac{7.78}{10^6}\left(\frac{100}{10}\right)^2 \sqrt{\frac{12\left(\frac{300{,}000}{100}\right)10}{0.625}}$$
$$= 0.591\ \text{s}$$

Determine C_s per ASCE/SEI 7-10, Section 12.8.1.1 with I/R = 1.0

$T_i \le T_S \Rightarrow C_s = S_{DS} = 0.733$
Base shear, V, = 0.733(300) = 219.9 kip
Per ASCE/SEI 7-10, Section 12.8.3, for T = 0.591 s, k = 1.045
Centroid for a distributed mass cantilever structure = $[(k+1)/(k+2)]H$ = 67.16 ft
Overturning moment = 219.9(67.16) = 14,768 ft-kip

Determine Stresses at Base of Skirt

Axial stress = P/A = 300,000/(π(10)12(0.625) = 1274 lb/in.^{2}
Bending stress = M/S = 14,768(1,000)12/[(0.625)π(10(12))^2/4] = 25,072 lb/in.^{2}

Example Problem 2. AWWA D100-05 Section 13.4.3.4

Seismic Information

S_S = 0.162, S_1 = 0.077
Site Class C, F_a = 1.2, F_v = 1.7
S_{DS} = 0.130, S_{D1} = 0.087, T_L = 12 s
Risk Category IV
T_s = 0.674 s

Tank Information

Structure Period T_i = 3.88 s
Class 2 Material: A36 (F_y = 36 kip/in.^{2})
Skirt angle (from vertical) = 15 deg
Weight of tank and water, W_W = 4,379 kip
Weight of tank, tower, and water, W_T = 4,502 kip
KL/r = 50

Determine S_{ai} per AWWA D100-05, Section 13.2.7.2

$T_s < T_i \le T_L \Rightarrow S_{ai} = S_{D1}/T_i = 0.087/3.88 = 0.0225$

Determine Critical Buckling Acceleration (I_e/1.4 R_i = 1)

Per Section 13.4.3.4, $A_i = S_{ai}$ for critical buckling check (A_i in AWWA D100-05 is the same as C_s in ASCE/SEI 7)
A_i = 0.0225

Lateral Displacement Caused by S_{ai} (P-Δ)

The final deflected position of the water centroid is an iterative process and must account for the additional moment applied to the structure because of the P-Δ effect. The deflection from the critical buckling deflection is equal to 3.89 in.

Check Skirt at Base of Tower

Seismic overturning moment at base of tower without P-Δ = 11,928 ft-kip (includes mass of tower).

Input Data for ASME Section VIII, Div. 2 Buckling Checks (Paragraph 4.4)

Input Values		
COURSE =	**Skirt**	
t = thickness of vessel section =	0.625	in
H_T = top elevation of course =	120	in
H_B = bottom elevation of course =	0	in
D_o = outer diameter of vessel section =	120	in
E_y = material modulus of elasticity =	29000000	psi
S_y = material yield strength =	36000	psi
P_{ext} = external pressure =	0.000	psi
f_a = axial comp membrane stress from axial load =	1274	psi
f_b = axial comp membrane stress from bending =	25072	psi
V = net section shear force =	219900	lbs
V_{phi} = applied shear force angle =	90	deg.
C_m = coefficient =	1	0.85, 1.0, or 0.6 - 0.4 (M_1 / M_2)
K_u = effective length factor =	2.1	Free-Fixed
L_u = maximum laterally unbraced length =	1200	in
L = design length vessel section for external pressure =	120	in
L = design length vessel section for axial compression =	120	in
FS = Input Factor of Safety =	**1.0**	
Calculated Values		
R_o = outter radius of shell section =	60	in
R = radius to centerline of shell =	59.6875	in
R_m = vessel mean radius =	59.6875	in
r = radius of gyration of cyl = $(1/4)(D_o^2+D_i^2)^{0.5}$ =	42.2	in
A = Cross sectional area of cylinder =	234.4	in^2
f_q = axial compressive membrane stress = $P\pi D_i^2/4A$ =	0.0	psi

Seismic overturning moment at base of tower with P-Δ = 11,928 ft-kip + 4,379 kip × 3.89 in./12 in. per ft = 13,348 ft-kip

Area of skirt = $\pi(26 \times 12)(0.625)$ = 612.6 in.^{2}

Section modulus of skirt = $\pi(26 \times 12)^2/4 \times 0.625$ = 47,784 in.^{3}

Skirt stress caused by axial load = 4,502(1,000)/(612.6 × cos 15) = 7,608 lb/in.^{2}

Skirt stress caused by moment = 13,348(12)(1000)/(47,784 × cos 15) = 3,470 lb/in.^{2}

Determine Critical Buckling Stress

R = 13 × 12/cos 15 = 161.5 in.

t/R = 0.625/161.5 = 0.0039

For Class 2 material, KL/r = 50, and t/R = 0.0039, determine allowable axial compressive stress, F_a, from Table 13 of AWWA D100-05.

F_a = 9882 lb/in.^{2}

Per AWWA D100-05, Section 13.4.3.4,

Critical buckling stress = $2F_a$ = 19,764 lb/in.^{2}

For Class 2 material and t/R = 0.0039, determine allowable bending compressive stress, F_b, from Table 11 of AWWA D100-05.

$F_b = F_L$ = 10,380 lb/in.^{2}

Per AWWA D100-05, Section 13.4.3.4,

Critical bending stress = $2F_b$ = 20.760 lb/in.^{2}

Check unity per AWWA D100-05, Section 3.3.1

7,608/19,764 + 3,470/20,760 = 0.552 ≤ 1.0 OK

C15.7.10.7 Concrete Pedestal (Composite) Tanks. A composite elevated water-storage tank is composed of a welded steel tank for watertight containment, a single-pedestal concrete support structure, a foundation, and accessories. The lateral load-resisting system is a load-bearing concrete shear wall. Because the seismic provisions in ACI 371R-98 are based on an older edition of ASCE/SEI 7, appropriate bridging equations are provided in Section 15.7.10.7.

C15.7.11 Boilers and Pressure Vessels. The support system for boilers and pressure vessels must be designed for the seismic forces and displacements presented in the standard. Such design must include consideration of the support, the attachment of the support to the vessel (even if "integral"), and the body of the vessel itself, which is subject to local stresses imposed by the support connection.

C15.7.12 Liquid and Gas Spheres. The commentary in Section C15.7.11 also applies to liquid and gas spheres.

C15.7.13 Refrigerated Gas Liquid Storage Tanks and Vessels. Even though some refrigerated storage tanks and vessels, such as those storing liquefied natural gas, are required to be designed for ground motions and performance goals in excess of those found in the standard, all such structures must also meet the requirements of this standard as a minimum. All welded steel refrigerated storage tanks and vessels must be designed in accordance with the requirements of the standard and the requirements of API 620.

4.4.12 Combined Loadings and Allowable Compressive Stresses (continued)

b) Axial Compressive Stress Acting Alone - The allowable axial compressive membrane stress of a cylinder subject to an axial compressive load acting alone, F_{xa}, is computed by following equations.

1) For $lamda_c \leq 0.15$ (Local Buckling)

F_{xa} = min [F_{xa1}, F_{xa2}] (4.4.61)

$F_{xa1} = S_y / FS$ for $D_o / t \leq 135$ (4.4.62)

$F_{xa1} = 466\ S_y / [(331 + D_o / t)\ FS]$ for $135 < D_o / t < 600$ (4.4.63)

$F_{xa1} = 0.5\ S_y / FS$ for $D_o / t \geq 600$ (4.4.64)

$F_{xa2} = F_{xe} / FS$ (4.4.65)

where: $F_{xe} = C_x\ E_y\ t / D_o$ (4.4.66)

C_x = min[409 c / [389 + D_o / t], 0.9] for $D_o / t < 1247$ (4.4.67)

$C_x = 0.25\ c$ for $1247 \leq D_o / t \leq 2000$ (4.4.68)

c = 2.64 for $M_x \leq 1.5$ (4.4.69)

$c = 3.13 / M_x^{0.42}$ for $1.5 < M_x < 15$ (4.4.70)

c = 1.0 for $M_x \geq 15$ (4.4.71)

$M_x = L / (R_o\ t)^{1/2}$ (4.4.124)

where L is the design length of a vessel section between lines of support

D_o / t = 192.00 135 < Do / t < 600

$M_x = L / (R_o\ t)^{1/2}$ = 19.60 > 15 c = 1.0000

Do / t < 1247 Cx = min [409 c / [389 + Do / t], 0.9] = 0.7039587 (4.4.67)

$F_{xe} = C_x\ E_y\ t / D_o$ = 106,327 psi (4.4.66)

Calculate Fxa_1			Calculate Fxa_2			
Fic = 466*Sy/(331+Do/t)	32076	psi	Fic =	106,327	psi	
Use Input FS =	1.00		Use Input FS =	1.0		
Fxa1 = 466*Sy/((331+Do/t)*FS)	32076	psi	Fxa_2 = Fxe / FS =	106,327	psi	(4.4.65)

Calculate Fxa

Fxa = min[F_{xa1}, F_{xa2}] = 32,076 psi (4.4.61)

$lambda_c$ = (K) (L_u) / [(pi)(r_g)] [(F_{xa}) (FS) / E]$^{0.5}$ = 0.6321 0.15 < lamda < 1.147

2) For $lamda_c > 0.15$ and $K_uL_u / r_g < 200$ (Column Buckling)

$F_{ca} = F_{xa}\ [1 - 0.74\ (lamda_c - 0.15)]^{0.3}$ for $0.15 < lamda_c \leq 1.147$ (4.4.72)

$F_{ca} = 0.88\ F_{xa} / (lamda_c)^2$ for $lamda_c \geq 1.147$ (4.4.73)

$K\ L_u / r_g$ = 59.7 < 200

lamda > 0.15 and K Lu / r < 200 therefore:

Fca = Fxa [1 - 0.74 (lamda - 0.15)]^0.3 = 28,100 psi

4.4.12 Combined Loadings and Allowable Compressive Stresses (continued)

c) Compressive Bending Stress - The allowable axial compressive membrane stress of a cylindrical shell subject to a bending moment acting across the full circular cross section F_{ba}, is computed using the following equations.

$F_{ba} = F_{xa}$	for $135 \le D_o / t \le 2000$	(4.4.74)
$F_{ba} = 466\ S_y / [(331 + D_o / t)\ FS]$	for $100 \le D_o / t < 135$	(4.4.75)
$F_{ba} = 1.081\ S_y / FS$	for $D_o / t < 100$ and $y \ge 0.11$	(4.4.76)
$F_{ba} = (1.4 - 2.9\ y)\ S_y / FS$	for $D_o / t < 100$ and $y < 0.11$	(4.4.77)
where: $y = S_y\ D_o / E_y\ t$		(4.4.78)

$D_o / t =$ 192

$y = S_y\ D_o / E_y\ t =$ 0.2383 Fic = Fxa

Fic = 32,076 psi

$D_o / t =$ 192.000 > 135 (see Sect. 3.1.1) Use Input FS = 1.0

Fba = Fxa = 32,076 psi

d) Shear Stress - The allowable shear stress of a cylindrical shell, F_{va}, is computed using the following equations.

$F_{va} = n_v\ F_{ve} / FS$		(4.4.79)
where: $F_{ve} = a_v\ C_v\ E\ t / D_o$		(4.4.80)
$C_v = 4.454$	for $M_x \le 1.5$	(4.4.81)
$C_v = (9.64 / M_x^2)\ (1 + 0.0239\ M_x^3)^{1/2}$	for $1.5 < M_x < 26$	(4.4.82)
$C_v = 1.492 / (M_x)^{1/2}$	for $26 \le M_x < 4.347\ D_o / t$	(4.4.83)
$C_v = 0.716\ (t / D_o)^{1/2}$	for $M_x \ge 4.347\ D_o / t$	(4.4.84)
$a_v = 0.8$	for $D_o / t \le 500$	(4.4.85)
$a_v = 1.389 - 0.218 \log_{10} (D_o / t)$	for $D_o / t > 500$	(4.4.86)
$n_v = 1.0$	for Fve / Sy < 0.48	(4.4.87)
nv = 0.43 Sy / Fve + 0.1	for 0.48 < Fve / Sy < 1.7	(4.4.88)
nv = 0.6 Sy / Fve	for Fve / Sy > 1.7	(4.4.89)

$D_o / t =$ 192.000

$M_x = L / (R_o\ t)^{0.5} =$ 19.596

1.5 < Mx < 26 Cv = (9.64/Mx^2)*(1+0.0239*Mx^3)^0.5 = 0.3376

Do / t < 500 av = 0.8000

$F_{ve} = a_v\ C_v\ E\ t / D_o =$ 40,793 psi

Fve / Sy = 1.13 0.48 < Fve / Sy < 1.7

nv = 0.43 Sy / Fve + 0.1 = 0.4795

$F_{ic} =$ 19,559 psi

Use Input FS = 1.000

$F_{va} = n_v\ F_{ve} / FS =$ 19,559 psi

4.4.12 Combined Loadings and Allowable Compressive Stresses (continued)

i) Axial Compressive Stress, Compressive Bending Stress, and Shear - the allowable compressive stress for the combination of uniform axial compression, axial compression due to bending, and shear in the absence of hoop compression.

Let $K_s = 1 - (f_v / F_{va})^2$ (4.4.105)

For 0.15 < $(\text{lamda})_c$ < 1.2

$\text{lamda}_c =$ 0.63 (Section 3.2) **0.15 < lamda < 1.2 OK**

$f_a / (K_s F_{ca}) + (8/9)(\text{delta}) f_b / (K_s F_{ba}) \leq 1.0$ $\quad f_a / (K_s F_{ca}) \geq 0.2$ (4.4.112)

$f_a / (2 K_s F_{ca}) + (\text{delta}) f_b / (K_s F_{ba}) \leq 1.0$ $\quad f_a / (K_s F_{ca}) < 0.2$ (4.4.113)

$K_s = 1 - (f_v / F_{va})^2 =$ 0.9977

$F_e = (\text{pi})^2 E / [K L_u / r]^2 =$ 80,287 psi

$\text{delta} = C_{m /} [1 - f_a FS / F_e] =$ 1.0161

$f_a / (K_s F_{ca}) =$ 0.045443 < 0.2

fa /(2 Ks Fca) + (delta) fb /(Ks Fba) =	**0.82**	**< 1.0**	***OK!***

C15.7.14 Horizontal, Saddle-Supported Vessels for Liquid or Vapor Storage. Past practice has been to assume that a horizontal, saddle-supported vessel (including its contents) behaves as a rigid structure (with natural period, T, less than 0.06 s). For this situation, seismic forces would be determined using the requirements of Section 15.4.2. For large horizontal, saddle-supported vessels (length-to-diameter ratio of 6 or more), this assumption can be unconservative, so Section 15.7.14.3 requires that the natural period be determined assuming the vessel to be a simply supported beam.

REFERENCES

American Concrete Institute (ACI). (2006), "Seismic design of liquid-containing concrete structures." *ACI 350.3-06*.

American Institute of Steel Construction (AISC). (1986). *Load and Resistance Factor Design Specification for Structural Steel Buildings.*

Anderson, P. F. (1966). "Temperature stresses in steel grain storage tanks." *Civil Engineering* 36(1), 74.

Chandrasekaran, A. R., and Jain, P. C. (1968). "Effective live load of storage materials under dynamic conditions." *Indian Concrete J.* (Bombay), 42(9), 364–365.

Chandrasekaran, A. R., Saini, S. S., and Jhamb, I. C. (1968). "Live load effects on dynamic behavior of structures." *J. of the Institution of Engineers* (India), 48, 850–859.

Federal Emergency Management Agency (FEMA). (2005). "Seismic considerations for steel storage racks located in areas accessible to the public." *FEMA 460*, Building Seismic Safety Council, National Institute of Building Sciences, Washington, D.C.

Kaups, T., and Lieb, J. M. (1985). *A Practical Guide for the Design of Quality Bulk Storage Bins and Silos*, Chicago Bridge & Iron Company, Plainfield, Ill.

Kilic, S., and Sozen, M. (2003). "Evaluation of effect of August 17, 1999, Marmara earthquake on two tall reinforced concrete chimneys." *ACI Struct. J.*, 100(3).

Malhotra, P. K. (2005). "Sloshing loads in liquid-storage tanks with insufficient freeboard." *Earthquake Spectra*, 21(4), 1185–1192.

Malhotra, P. K. (2006). "Earthquake induced sloshing in cone and dome roof tanks with insufficient freeboard." *J. of Structural Engineering International*, IABSE, 16(3), 222–225.

Malhotra, P. K., Wenk, T., and Wieland, M. (2000). "Simple procedure for seismic analysis of liquid-storage tanks." *J. of Structural Engineering International*, IABSE, 10(3), 197–201.

Miller, C. D., Meier, S. W., and Czaska, W. J. (1997). "Effects of internal pressure on axial compressive strength of cylinders and cones," paper presented at Structural Stability Research Council Annual Technical Meeting, June.

Trahair, M. S., Abel, A., Ansourian, P., Irvine, H. M., and Rotter, J. M. (1983). *Structural Design of Steel Bins for Bulk Solids.* Australian Institute of Steel Construction, Ltd., Sydney.

Troitsky, M. S. (1990). *Tubular Steel Structures—Theory and Design*. The James F. Lincoln Arc Welding Foundation.

U.S. Department of Energy. (1963). "Nuclear reactors and earthquakes." *TID-7024*, U.S. Atomic Energy Commission.

Veletsos, A. S. (1974). "Seismic effects in flexible liquid-storage tanks." In *Proceedings of the Fifth World Conference on Earthquake Engineering*, Rome, Italy, 630–639.

Wozniak, R. S., and Mitchell, W. W. (1978). "Basis of seismic design provisions for welded steel oil storage tanks." Presented at the Session on Advances in Storage Tank Design, American Petroleum Institute, Refining, 43rd Midyear Meeting, Toronto, Canada, May 9.

Minimum Design Loads for Buildings and Other Structures

The following figures were inadvertently omitted from the Commentary to Chapter 15.

CHAPTER C15 FIGURES
SEISMIC DESIGN REQUIREMENTS FOR NONBUILDING STRUCTURES

Figure C15.1-1 Steel multilegged water tower

Figure C15.1-2 Steel pipe rack

Figure C15.1-3 Heat recovery steam generators

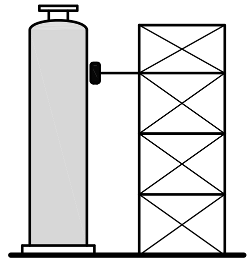


Figure C15.1-4 Multiple lateral supports

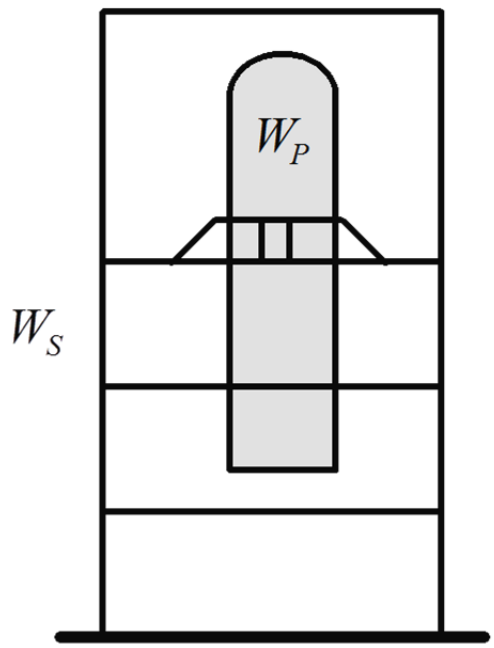


Figure C15.1-5 Unusual support of dead weight

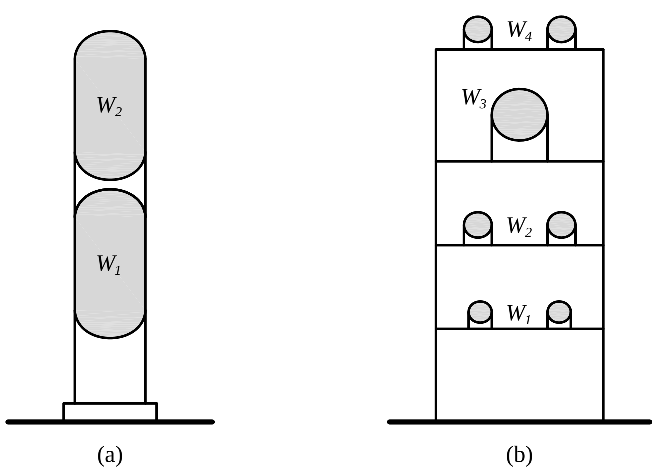


Figure C15.1-6 Mass irregularities

Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-10)

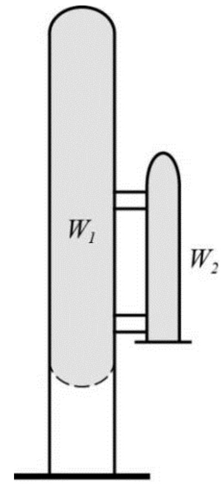


Figure C15.1-7 Torsional irregularity

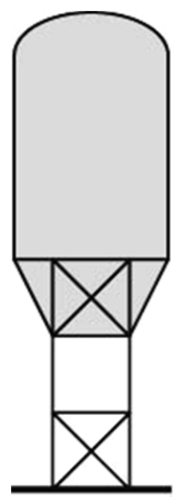


Figure C15.1-8 Soft-story irregularity

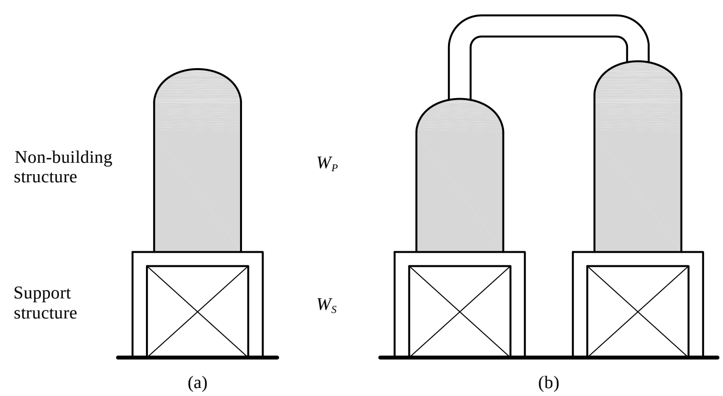


Figure C15.1-9 Coupled system

Figure C15.5-1 Merchandise restrained by netting

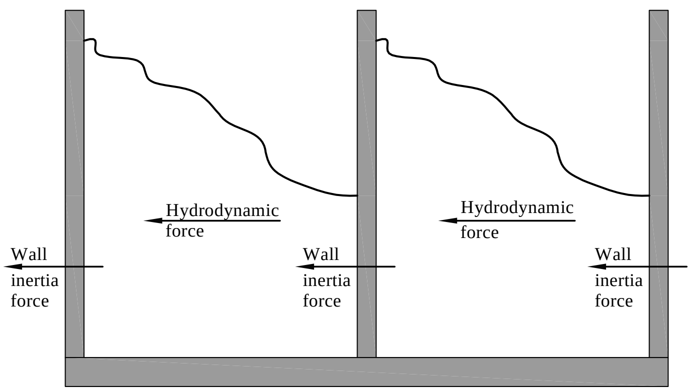


Figure C15.6-1 Wall forces

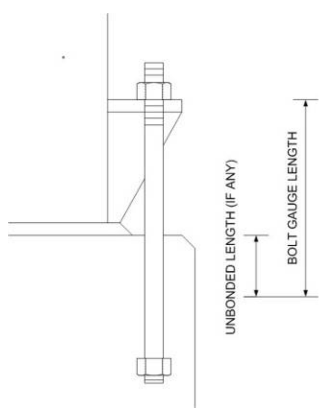


Figure C15.7-1 Bolt Gauge Length

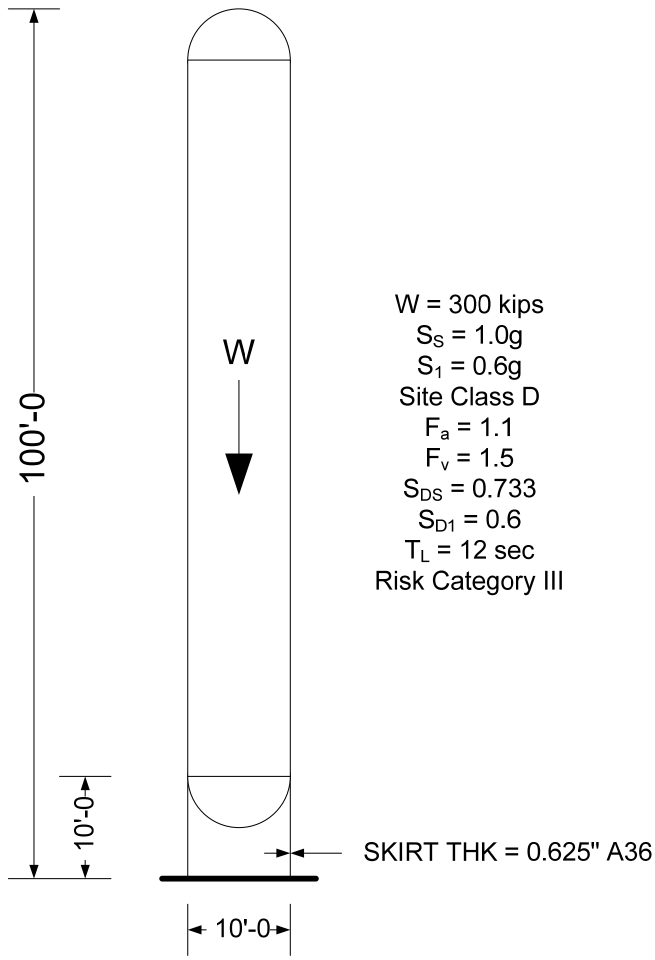


Figure Ex- 1 Vertical Vessel

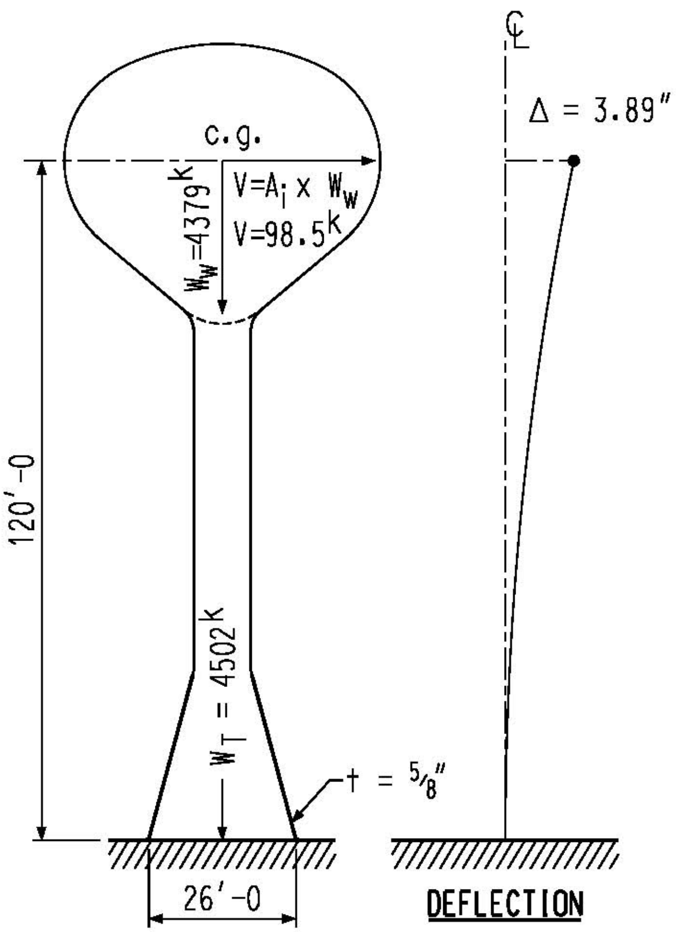
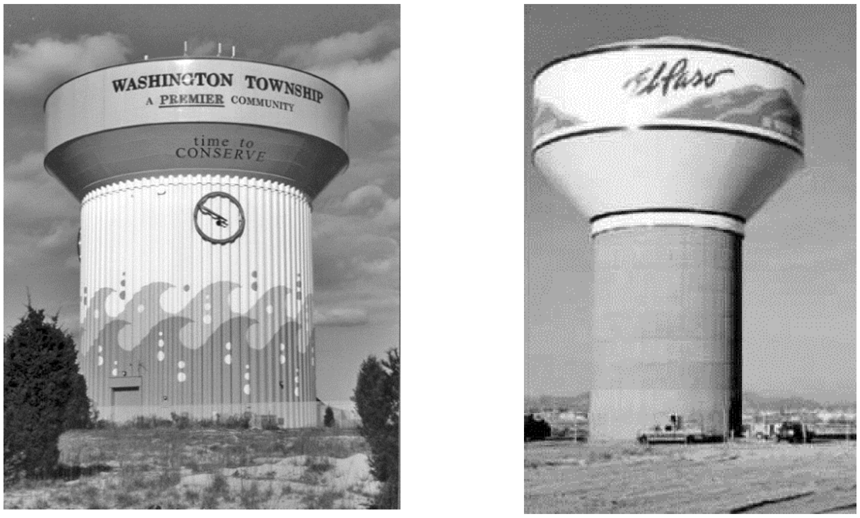


Figure Ex- 2 Single Pedestal Elevated Water Tank

Figure C15.7-2 Multileg braced steel tank

Figure C15.7-3 Small-diameter single-pedestal steel tank



(a) Steel (b) Concrete

Figure C15.7-4 Large-diameter single-pedestal tank

CHAPTER C16
SEISMIC RESPONSE HISTORY PROCEDURES

C16.1 LINEAR RESPONSE HISTORY PROCEDURE

The standard does not require the use of linear response history analysis for design of a structure. The use of such analysis may be useful in validation of the results from the analysis methods presented in Chapter 12, or as a step in a series of analyses that culminate in a nonlinear response history analysis. Although not commonly used in the past to design conventional structures, this technique is seeing increased use in the design of some structures that neither contain damping systems nor are base isolated.

The purpose of the linear response history procedure is to determine design forces for structural components and to compute displacements and story drifts, which must be within the limits specified by Table 12.12-1. In this sense, the linear response history procedure shares the same force-based philosophy of the equivalent lateral force (ELF) procedure and the modal response spectrum (MRS) analysis procedure (specified in Chapter 12). Response history analysis offers several advantages over modal response spectrum analysis: it is more accurate mathematically, signs of response quantities (such as tension or compression in a brace) are not lost as a result of the combination of modal responses, and story drifts are computed more accurately. The principal disadvantages of response history analysis are the need to select and scale an appropriate suite of ground motions, and the necessity to perform analysis for several (usually seven) such motions. See Section C16.1.3 and C16.1.4 for discussion of ground motion selection and scaling techniques.

C16.1.1 Analysis Requirements. In response history analysis, the seismic hazard is characterized by a number of ground acceleration records. Using these records and a detailed mathematical model of the structure, nodal displacements and component forces are computed, step by step, by solving the equations of motion. There are two basic approaches for solving these equations. In the first approach, called direct analysis, all the equilibrium equations for the entire system are solved simultaneously in each time step.

In the second approach, called modal analysis, the equilibrium equations are transformed, by change of coordinates, into a number of uncoupled single-degree-of-freedom (SDOF) systems, one for each mode.

Where modal analysis uses the full set of mode shapes and the damping ratios in each mode are identical to those obtained from the equations of motion used in the direct analysis, the two approaches produce identical results. A distinct advantage of the modal analysis approach is that a limited number of modes may be used to produce reasonably accurate results. Although some accuracy is sacrificed where fewer modes are used, the computer resources required to perform the analysis are significantly less than those required for direct analysis. The number of modes required for a reasonably accurate analysis is discussed in Section C12.9.1.

The damping matrix plays a very important role in the solution of the equations of motion. The classical modal analysis procedure, discussed above, is not applicable for structures consisting of subsystems with very different levels of damping (Chopra 2007), for example, base-isolated structures, structures with energy-dissipating devices, and soil-structure interaction (when including both the structure and supporting soil in the analysis). This is a limitation of the uncoupled classical damping matrix. For nonclassical damping (coupled matrix), an alternative modal analysis approach, significantly more computationally demanding, is to include the damping term in the eigenvalue problem (Veletsos and Ventura 1986). The direct analysis procedure has the advantage that the nonclassical damping matrix can be employed in the solution. The designer should be aware of the limitations of each solution method to represent the dynamic response of a structure.

C16.1.2 Modeling. The mathematical model used for linear response history analysis is usually identical to that used for modal response spectrum analysis, and it often reflects a preliminary design developed using the ELF procedure. The main modeling difference between response history analysis and modal response spectrum analysis is that the inherent damping (taken as 5% of critical) is included in the design response spectrum for modal response spectrum analysis, whereas it must be assigned explicitly for response history analysis.

In the modal analysis approach, damping is simply assigned to each mode that is included in the response (Wilson and Penzien 1970). Although not specified in the standard, the damping used for each mode should be 5% of critical for consistency with the design response spectrum.

Linear response history analysis requires an explicit damping matrix. However, such a matrix cannot be formed from first principles; it is common to use a damping matrix that is proportional to the mass, the stiffness, or a linear combination of the two:

$$C = \alpha M + \beta K \qquad \text{(C16.1-1)}$$

where C is the damping matrix, M is the mass matrix, K is the stiffness matrix, and α and β are scalar constants of proportionality for each system forming the analytical model. Such damping is often referred to as Rayleigh damping.

The proportionality constants are determined as follows:

$$\begin{bmatrix}\alpha\\ \beta\end{bmatrix} = 2\begin{bmatrix}1/\omega_a & \omega_a\\ 1/\omega_b & \omega_b\end{bmatrix}^{-1}\begin{bmatrix}\xi_a\\ \xi_b\end{bmatrix} \qquad \text{(C16.1-2)}$$

where ξ_a and ξ_b are the desired damping ratios at any two system circular frequencies, ω_a and ω_b, where $\omega_b > \omega_a$. It is common,

but not necessary, for the two specified frequencies to correspond to two of the system's lower natural frequencies (such as the first and third mode frequencies).

If both damping values are the same ($\xi = \xi_a = \xi_b$), which is usually the case, the mass and stiffness proportionality constants may be determined as follows:

$$\alpha = \xi \frac{2\omega_a \omega_b}{\omega_a + \omega_b}$$
$$\beta = \xi \frac{2}{\omega_a + \omega_b} \qquad \text{(C16.1-3)}$$

The advantage of Rayleigh damping is that it is simple to implement because all the analyst has to do is to specify the two proportionality constants α and β, and these can be established using Eq. C16.1-2 given the two desired damping ratios and corresponding frequencies. The disadvantage is that the damping ratios increase with frequency and may cause the higher mode contributions to response to be overdamped. This effect is shown in Figure C16.1-1, where the damping ratios ξ have been set at 0.05 at frequencies of 4.2 and 12.5 radians per second. The damping at all other frequencies is given by the curve marked "total." For frequencies above approximately 32 radians per second, the damping is greater than 10% of critical and may be excessive.

C16.1.3 Ground Motion. One of the most demanding aspects of response history analysis is the selection and scaling of an appropriate suite of ground motions (Anderson and Bertero 1987). It is considered appropriate to select records that have magnitudes, fault distances, source mechanisms, and soil conditions that are characteristic of the site. This selection poses quite a challenge even for sites in the western United States, where numerous records from large-magnitude earthquakes are available; it is virtually impossible in the central and eastern United States, where there are no recorded ground motions from large-magnitude events. The website for the Pacific Earthquake Engineering Research Center (PEER) provides a large number of ground motion acceleration records that may be used in response history analysis. In addition to the ground motions, the PEER site provides detailed background information on the source characteristics of the ground motions and on the instrument and site characteristics of the particular station that recorded the acceleration record.

Because of the scarcity of available recorded motions, use of simulated ground motions is permitted. To this end, available records may be modified for site distance and soil conditions. Such modification is considered part of the ground motion selection.

The standard requires that at least three ground motions (or ground motion pairs, in the case of three-dimensional analysis) be used, and it provides an incentive for using at least seven motions (as discussed in Section C16.1.4).

The scaling technique specified in Sections 16.1.3.1 and 16.1.3.2 is one of several that have been proposed. See Shome and Cornell (1998), Shome et al. (1998), Somerville et al. (1998), Mehrain and Naeim (2003), and Iervolino and Cornell (2005) for background on ground motion selection and scaling.

C16.1.3.1 Two-Dimensional Analysis. This scaling method begins with ground motions that have been selected (and modified as necessary) to have magnitude, distance, and site conditions compatible with the maximum considered earthquake. The 5% damped pseudoacceleration response spectra for these records are scaled for consistency with the design ground motion spectrum shown in Fig. 11.4-1. For two-dimensional analysis, the ground motion spectra must be scaled such that the average of the spectra is not less than the design spectrum in the period range from 0.2T to 1.5T, where T is the fundamental period of vibration of the structure being designed. The short period of the range (0.2T) is set to capture higher mode response, and the long period of the range (1.5T) is set to allow for period lengthening that would be associated with inelastic response.

C16.1.3.2 Three-Dimensional Analysis. Approaches to scaling ground motions for three-dimensional analysis are similar to those for two-dimensional analysis. The two orthogonal components within each pair must have the same scale factor, but the individual pairs may have different scale factors. Within 3 mi

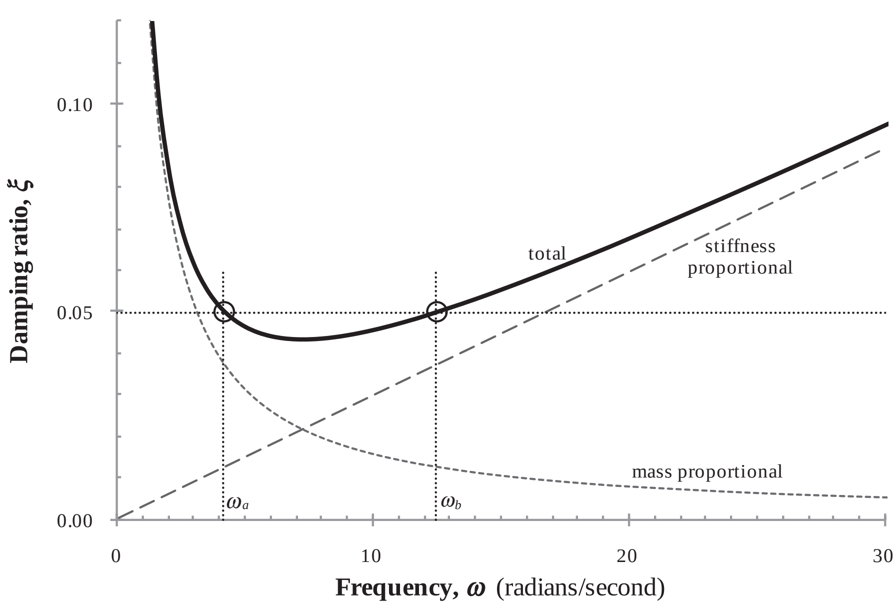


FIGURE C16.1-1 Example of Rayleigh Damping

(5 km) of an active fault, ground motion components selected to represent fault-normal and fault-parallel directions are required to be included in the analysis. For certain structures, the response under both horizontal and vertical ground motions should be considered. Vertical ground motion spectra are not readily available. Section 12.4.2.2 provides procedures for estimating vertical seismic forces.

Given a set of appropriate ground motions, there are an infinite number of scaling factors that may be applied to the individual motions to meet the requirements of Sections 16.1.3.1 and 16.1.3.2. Thus, two analysts, working with the same set of ground motions, are likely to produce a different set of scale factors. Although this difference in scaling would have little effect in linear analysis, it may lead to vastly different results in nonlinear analysis. For this reason, the process of selection and scaling of ground motions should be included in the design review (Section 16.2.5 of the standard) that is required wherever nonlinear response history analysis is used.

Both amplitude scaling and spectral matching procedures can be used to satisfy the scaling technique specified in Sections 16.1.3.1 and 16.1.3.2. Both procedures provide reasonable estimates of mean response for the individual response parameters. Spectral matching can provide mean estimates with a smaller suite of motions, although seven suites are still required, as outlined in Section 16.1.4. Neither scaling approach, however, is adequate to give an accurate estimate of the variability, although amplitude scaling gives a better understanding of the potential variability than spectral matching.

C16.1.4 Response Parameters. The responses derived from the response history analysis are multiplied by the importance factor, I_e, to provide enhanced strength and stiffness for more important facilities and are divided by R to account for inelastic behavior. For consistency with the ELF and MRS analysis procedures, the displacements computed from the response histories that have been modified by I_e/R should be multiplied by C_d/I_e to obtain the displacement histories for computing the story drift histories.

The base shear typically is computed from component elastic forces. A slightly different shear would be computed from the total inertial forces; the difference is caused by damping. Although the results of MRS analysis must be scaled up such that the corresponding base shear is not less than 85% of the base shear that would be computed from an ELF analysis (see Section C12.9.4) for strength design forces, the scaling for linear response history analysis considers only 100% of the applicable minimum base shear coefficient. Similar but slightly different provisions (including the 85% multiplier) are prescribed for scaling of drifts.

If seven or more ground motions are used, the design values may be taken as the average of the scaled values from the response history analysis. This requirement provides some difficulty for components for which the capacity depends on multiple values. For a column, for example, both the axial force and the concurrent bending moment are needed to compare demand and capacity. In that instance, if seven or more ground motions are used, the column is deemed suitable if the average of the seven peak demand-to-capacity ratios for the column is less than 1.0. Where fewer than seven ground motions are used, the column is deemed suitable if the maximum demand-to-capacity ratio is less than 1.0.

The direction of loading requirements of Section 12.5 and the modeling requirements of Section 12.7 apply to response history analysis. Accidental torsion, amplification of accidental torsion, or detailed consideration of P-delta effects should be included in a manner consistent with the requirements of Section 12.9 (see discussion in C12.9). Whether or not gravity loads should be applied simultaneously with the ground motion records depends on whether the member stiffness is updated during each time step or is initially set constant by a vertical load combination.

C16.2 NONLINEAR RESPONSE HISTORY PROCEDURE

Nonlinear response history analysis is not used as part of the normal design process for conventional structures. In some cases, however, nonlinear analysis is recommended, and in certain cases required, to obtain a more realistic assessment of structural response and verify the results of simpler methods of analysis. Such is the case for systems with friction-based passive energy dissipation devices, nonlinear viscous dampers, seismically isolated systems, self-centering systems, or systems that have components with highly irregular force-deformation relationships.

The principal aim of nonlinear response history analysis is to determine if the computed deformations of the structure are within appropriate limits. Strength requirements for the designated lateral load-resisting elements do not apply because element strengths are established before the analysis. These initial strengths typically are determined from a preliminary design using linear analysis.

The nonlinear response history analysis may also provide useful information on the strength requirements for nonstructural components, which are often assumed to remain elastic in the analysis.

Where displacements computed from the nonlinear response history analysis are excessive, a typical remedy is to increase the stiffness of the structure, which is likely to affect the computed strength.

Nonlinear response history analysis offers several advantages over linear response history analysis, including the ability to model a wide variety of nonlinear material behaviors, geometric nonlinearities (including large displacement effects), gap opening and contact behavior, and nonclassical damping, and to identify the likely spatial and temporal distributions of inelasticity. Nonlinear response history analysis has several disadvantages, including increased effort to develop the analytical model; increased time to perform the analysis (which is often complicated by difficulties in obtaining converged solutions); sensitivity of computed response to system parameters; and the inapplicability of superposition to combine live, dead, and seismic load effects.

C16.2.1 Analysis Requirements. Nonlinear response history analysis is highly sensitive to the modeling assumptions and solution procedures used. Seemingly minor changes in assumptions of elastic stiffness and strength, hysteretic behavior, solution time step, and other parameters can significantly affect the indicated results. *NEHRP Technical Brief No. 4, Nonlinear Structural Analysis for Seismic Design* (Deierlein et al. 2010) provides guidance on the many decisions that must be made. Sensitivity analysis that explores the effect of modeling assumptions on predicted response is often a valuable tool to provide confidence in the analysis results. Other validation methods can include benchmarking analyses with alternative software and modeling structures with known behavior, such as laboratory tests, to confirm that reasonable solutions are obtained.

The mathematical model need only comply with the requirements of Section 12.7, which does not explicitly require

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accidental torsion to be modeled. It is generally accepted practice that when three-dimensional dynamic analysis is used in conjunction with the requirements of nonlinear response history procedure, the effects of accidental torsion are not explicitly modeled because they are assessed automatically. Although consideration of accidental torsion (and amplification) is usually associated with the uncertainty regarding the center of mass and stiffness, it also accounts for torsional ground motion and associated response effects and the increased torsional response associated with nonlinear behavior of the lateral force-resisting system. Because the nonlinear behavior is being modeled and torsional ground motion effects are deemed to be small for conventional building structures with regular plans, it is considered unnecessarily conservative to additionally include accidental torsion effects, especially using the "dynamic" approach discussed in Section C12.9.5. Furthermore, for complex 3-D nonlinear response history analyses, it is often considered impractical to require multiple analyses to address all possibilities of eccentricity.

C16.2.2 Modeling. Nonlinear response history analysis requires a mathematical description of the hysteretic behavior of those portions of the structure that are expected to exhibit inelastic behavior during an earthquake. Such models must reflect the expected material properties of the components and account for the following behavioral effects as appropriate:

1. Material overstrength and strain hardening,
2. Cyclic degradation of stiffness and strength,
3. In-cycle degradation of stiffness and strength (Applied Technology Council 2009),
4. Pinching,
5. Buckling, and
6. Axial-flexural-shear interaction.

Other effects inherent to a component (e.g., bolt slip, bond slip, gaps, sliding shear) represent inelastic actions as distinct elements (e.g., a nonlinear hinge spring to model a plastic hinge) using concentrated phenomenological force-deformation relationships. More exact analysis may be performed using distributed plasticity models by subdividing inelastic portions into a number of slices or fibers. This more exact approach is preferable but is more computationally demanding.

An inelastic three-dimensional analysis is particularly useful for buildings that are prone to torsional response in plan, even where the main seismic force-resisting systems resist loads predominantly in their own plane. If only two-dimensional software is available, a "pseudo" three-dimensional analysis may be performed (Mehrain and Naeim 2003).

In moment-resisting frames, the elastic and inelastic behavior of beam-column joint regions should be modeled explicitly. P-delta effects should be considered explicitly in the analysis, with gravity loads applied simultaneously. Nonstructural components also should be included in the model if it is expected that their stiffness and strength have a significant effect on the response (e.g., infill walls).

Nonlinear response history analysis requires that inherent damping be set for the structure. As for linear response history procedure, analysis is typically performed assuming inherent damping of 5% of critical. However, the use of 5% of critical for inherent damping may be quite unconservative where the model includes significant hysteretic damping elements. Some analysts and designers advocate the use of lower levels of inherent damping (perhaps 2% of critical), especially for steel frames, but there is no widespread agreement on this point.

The mechanism used to include inherent damping in the analytical model is critically important to the accuracy of the computed response. Most nonlinear analysis programs use a form of Rayleigh damping, wherein the damping matrix (used for direct integration of the equations of motion) is represented as a linear combination of the mass and stiffness matrices (see Section C16.1.2). If the damping matrix is based on the initial stiffness (elastic) of the system, artificially high damping forces may be generated by system yielding. In some cases, these forces can completely skew the computed response (Chrisp 1980, Carr 2004, Charney 2006, and Hall 2006). One method to counter this occurrence is to base the damping matrix on the mass and the instantaneous tangent stiffness. Where basing the damping on tangent stiffness, care must be taken so that the damping is not negative when the tangent stiffness is negative. Other approaches have been suggested, such as capped Rayleigh damping (Hall 2006), hysteretic damping (Charney 2006), or a combination of modal and Rayleigh damping (Powell 2010).

Three-dimensional analysis must be used where certain horizontal structural irregularities are present. For structures composed of two-dimensional seismic force-resisting elements connected by floor and roof diaphragms, the diaphragms should be modeled as semirigid in plane, particularly where the vertical elements of the seismic force-resisting system are of different types (such as moment frames and walls). Where structures are modeled in three dimensions, axial force–biaxial bending interaction should be considered for corner columns, nonrectangular walls, and other similar elements.

To properly capture the nonlinear dynamic response of structures where vertical dynamic response may have a significant influence on structural performance, it is at times necessary to include vertical mass in the mathematical model, even though vertical ground motions are not included in the analysis. Typically the vertical mass is only provided at column nodes, although it could be included at all vertical degrees of freedom, provided the vertical stiffness of members has been modeled properly. Numerical convergence problems caused by large oscillatory vertical accelerations have been noted (NEHRP 2010) where base rotations caused by wall cracking in fiber wall models are the primary source of vertical excitation. Sensitivity studies should be conducted as discussed in Section C16.2.1 when vertical mass is included in the mathematical model.

As mentioned previously, P-delta effects should be included where significant. The significance of P-delta effects on the overall response may be assessed by performing analyses with and without P-delta effects and comparing story drift response histories. Destabilizing effects of gravity loads are often manifested by accumulated residual deformations, and these deformations, if not controlled, can lead to dynamic instability of the structure.

C16.2.3 Ground Motion and Other Loading. Because linear superposition cannot be used with nonlinear analysis, each response history analysis must begin with an initial gravity load (sometimes called the initial ramping load), consisting of the expected dead load and live load. The live load may be as little as 25% of the unreduced design live load because multiple transient loads are unlikely to attain their maxima simultaneously. When determining behaviors such as uplift or column tension, the value of the 0.25 live load should be taken as zero when it results in a larger response of concern, which is consistent with the requirements in Section 2.3.2.

C16.2.4 Response Parameters. As discussed previously, the principal aim of nonlinear response history analysis is to determine deformation demands in structural and nonstructural

components for comparison with accepted limits. Where at least seven ground motions are used, the member and connection deformations may be taken as the average of the values computed from the analyses. If fewer than seven motions are used, the maximum values among all analyses must be used. It is very important to note, however, that assessment of deformations in this manner should not be done without careful inspection of the story displacement histories of each analysis. It is possible that the maximum displacement or drift may be completely dominated by the response from one ground motion, and such dominance, when caused by ratcheting (increasing deformations in one direction resulting in a high residual deformation), may be a sign of imminent dynamic instability. Where these kinds of dynamic instabilities are present, the analyst should attempt to determine the system characteristics that produce such effects. The ground motion that produces dynamic instability should not be replaced with one that does not.

C16.2.4.1 Member Strength. The seismic load effects, including the overstrength factor of Section 12.4.3, need not be assessed because linear combinations of load are not applicable in nonlinear analysis. Overstrength effects are evaluated directly because hysteretic force-deformation relationships are modeled explicitly, and the material properties so used include overstrength and strain hardening (as required by Section 16.2.2).

C16.2.4.2 Member Deformation. This section requires that member and connection deformations be assessed on the basis of tests performed for similar configurations.

C16.2.4.3 Story Drift. The 25% increase in allowable story drift is provided because the nonlinear analysis is generally more accurate than linear analysis and because member deformations are assessed explicitly.

C16.2.5 Design Review. As discussed previously, nonlinear response history analysis is quite complex, and the results may be strongly influenced by subtle changes in ground motion or system properties. Hence, such analysis must only be conducted by experienced professionals with training in engineering seismology, earthquake engineering, structural dynamics, stability, nonlinear analysis, and inelastic behavior of structures. Regardless of the level of expertise of the individual or individuals who perform the analysis and design, a design (peer) review of the structural system and the nonlinear analysis is required wherever the design is based on the nonlinear response history procedure.

REFERENCES

Anderson, J. A, and Bertero, V. V. (1987). "Uncertainties in establishing design earthquakes." *J. of Structural Engineering*, 113(8), 1709–1724.

Applied Technology Council. (2009). *The Effects of Strength and Stiffness Degradation on Seismic Response*, FEMA P440A, FEMA, Washington, D.C.

Carr, A. J. (2004). *Ruaumoko Users Manual*, Volume 1. Department of Civil Engineering, University of Canterbury, New Zealand.

Charney, F. A. (2006). "Unintended consequences of modeling damping in structures: Rayleigh damping." *Structures Congress 2006: 17th Analysis and Computation Conference*, ASCE, Reston, Va.

Chopra, A. K. (2007). *Dynamics of Structures: Theory and Applications to Earthquake Engineering*, Prentice Hall, Upper Saddle River, NJ, 876.

Chrisp, D. J. (1980). "Damping models for inelastic structures." Master of Engineering Report, University of Canterbury, New Zealand.

Deierlein, G. G., Reinhorn, A. M., and Willford, M. R. (2010). "Nonlinear structural analysis for seismic design." *NEHRP Seismic Design Technical Brief No. 4*. Produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, Md, NIST GCR 10-917-5.

Hall, J. F. (2006). "Problems encountered from the use (or misuse) of Rayleigh damping." *Earthquake Engineering and Structural Dynamics*, 25(5), 525–545.

Iervolino, I., and Cornell, A. (2005). "Record selection for nonlinear seismic analysis of structures." *Earthquake Spectra*, 21(3), 685–714.

Mehrain, M., and Naeim, F. (2003). "Exact three-dimensional linear and nonlinear seismic analysis of structures using two-dimensional models." *Earthquake Spectra*, 19(4), 897–912.

NEHRP Consultants Joint Venture. (2010). *Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Factors*, Produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, Md., NIST GCR 10-917-8.

Powell, G. H. (2010). *Modeling for Structural Analysis: Behavior and Basics*. Computers and Structures, Berkeley, CA, 365.

Shome, N., and Cornell, A. (1998). "Normalization and scaling accelerograms for nonlinear analysis." *Proceedings of the 6th U.S. National Conference on Earthquake Engineering.* Earthquake Engineering Research Institute, Oakland, CA.

Shome, N., Cornell, A., Bazzurro, P., and Carballo, J. E. (1998). "Earthquakes, records, and nonlinear response," *Earthquake Spectra*, 14(3), 469–500.

Somerville, P., Anderson, D., Sun, J., Punyamurthula, S., and Smith, N. (1998). "Generation of ground motion time histories for performance based seismic engineering." *Proceedings of the 6th U.S. National Conference on Earthquake Engineering*, Earthquake Engineering Research Institute, Oakland, CA.

Velestos, A. S., and Ventura, C. E. (1986). "Modal analysis of non-classically damped linear systems." *Earthquake Engineering and Structural Dynamics*, 14, 217–243.

Wilson, E. L., and Penzien, J. (1970). "Evaluation of orthogonal damping matrices." *International Journal for Numerical Methods in Engineering*, 4, 5–10.

OTHER REFERENCES (NOT CITED)

Naeim, F., Alimoradi, A., and Pezeshk, S. (2004). "Selection and scaling of ground motion time histories for structural design using genetic algorithms." *Earthquake Spectra*, 20(2), 413–426.

Vamvatsikos, D. (2002). "Seismic performance, capacity and reliability of structures as seen through incremental dynamic analysis." Ph.D. Dissertation, Department of Civil and Environmental Engineering, Stanford University, Palo Alto, Calif.

Wilson, E. L. (2004). *Static and Dynamic Analysis of Structures*, 4th Ed. Computers and Structures, Inc., Berkeley, CA.

Wilson, E. L., Der Kiureghian, A., and Bayo, E. (1981). "A replacement for the SRSS method in seismic analysis." *Earthquake Engineering and Structural Dynamics*, 9, 192.

Wilson, E. L., Yuan, M., and Dickens, J. (1982). "Dynamic analysis by direct superposition of Ritz vectors." *Earthquake Engineering and Structural Dynamics*, 10, 813–823.

CHAPTER C17
SEISMIC DESIGN REQUIREMENTS FOR SEISMICALLY ISOLATED STRUCTURES

C17.1 GENERAL

Seismic isolation, commonly referred to as base isolation, is a method used to substantially decouple the response of a structure from potentially damaging earthquake motions. This decoupling can result in response that is reduced significantly from that of a conventional, fixed-base building.

The potential advantages of seismic isolation and the recent advancements in isolation-system technology have led to the design and construction of a large number of seismically isolated buildings and bridges in the United States.

Design requirements for seismically isolated structures were first codified in the United States as an appendix to the 1991 *Uniform Building Code*, based on "General Requirements for the Design and Construction of Seismic-Isolated Structures" developed by the Structural Engineers Association of California State Seismology Committee. In the intervening years, those provisions have developed along two parallel tracks into the design requirements in Chapter 17 of the standard and the rehabilitation requirements in Section 9.2 of ASCE/SEI 41, *Seismic Rehabilitation of Existing Buildings*. The design and analysis methods of both standards are quite similar, but ASCE/SEI 41 permits more liberal design for the superstructure of rehabilitated buildings. The AASHTO (1999) *Guide Specification for Seismic Isolation Design* provides a systematic approach to determining bounding values of mechanical properties of isolators for analysis and design. Rather than addressing a specific method of seismic isolation, the standard provides general design requirements applicable to a wide range of possible seismic isolation systems. Because the design requirements are general, testing of isolation-system hardware is required to confirm the engineering parameters used in the design and to verify the overall adequacy of the isolation system. Use of isolation systems whose adequacy is not proved by testing is prohibited. In general, acceptable systems (a) remain stable when subjected to design displacements, (b) provide increasing resistance with increasing displacement, (c) do not degrade under repeated cyclic load, and (d) have quantifiable engineering parameters (such as force-deflection characteristics and damping).

The force-deflection behavior of isolation systems falls into four categories, as shown in Fig. C17.1-1, where each idealized curve has the same design displacement, D_D. A linear isolation system (Curve A) has an effective period independent of displacement, and the force generated in the superstructure is directly proportional to the displacement of the isolation system.

A hardening isolation system (Curve B) is soft initially (long effective period) and then stiffens (effective period shortens) as displacement increases. Where displacements exceed the design displacement, the superstructure is subjected to larger forces and the isolation system to smaller displacements than for a comparable linear system.

A softening isolation system (Curve C) is stiff initially (short effective period) and then softens (effective period lengthens) as displacement increases. Where displacements exceed the design displacement, the superstructure is subjected to smaller forces and the isolation system to larger displacements than for a comparable linear system.

The response of a purely sliding isolation system (Curve D) is governed by the friction force at the sliding interface. For increasing displacement, the effective period lengthens and loads on the superstructure remain constant. For isolation systems governed solely by friction forces, the total displacement caused by repeated earthquake cycles is highly dependent on the characteristics of the ground motion and may exceed the design displacement, D_D. Because such systems do not have increasing resistance with increasing displacement, the procedures of the standard cannot be applied, and use of the system is prohibited.

Chapter 17 provides isolator design displacements, shear forces for structural design, and other specific requirements for seismically isolated structures. All other design requirements, including loads (other than seismic), load combinations, allowable forces and stresses, and horizontal shear distribution, are the same as those for conventional, fixed-base structures.

C17.1.1 Variations in Material Properties. For analysis, the mechanical properties of seismic isolators generally are based on values provided by isolator manufacturers. Values of these properties should be in the range that accounts for natural variability and uncertainty, and variability of properties among isolators of different manufacturers. Examples may be found in Constantinou et al. (2007). Prototype testing is used to confirm the values assumed for design. Unlike conventional materials whose properties do not vary substantially with time, the materials used in seismic isolators have properties that generally vary with time. Because mechanical properties can vary over the life of a structure and the testing protocol of Section 17.8 cannot account for the effects of aging, contamination, scragging (temporary degradation of mechanical properties with repeated cycling), temperature, velocity effects, and wear, the designer must account for these effects by explicit analysis. One approach to accommodate these effects, introduced in Constantinou et al. (1999), is to use property modification factors. Information on variations in material properties of seismic isolators and dampers is reported in Constantinou et al. (2007).

C17.2 GENERAL DESIGN REQUIREMENTS

Ideally, most of the lateral displacement of an isolated structure is accommodated by deformation of the isolation system rather than distortion of the structure above. Accordingly, the seismic force-resisting system of the structure above the isolation system

is designed to have sufficient stiffness and strength to avoid large, inelastic displacements. Therefore, the standard contains criteria that limit the inelastic response of the structure above the isolation system. Although damage control is not an explicit objective of the standard, design to limit inelastic response of the structural system also reduces the level of damage that would otherwise occur during an earthquake. In general, isolated structures designed in accordance with the standard are expected:

1. To resist minor and moderate levels of earthquake ground motion without damage to structural elements, nonstructural components, or building contents and
2. To resist major levels of earthquake ground motion without failure of the isolation system, significant damage to structural elements, extensive damage to nonstructural components, or major disruption to facility function.

Isolated structures are expected to perform much better than fixed-base structures during moderate and major earthquakes. Table C17.2-1 compares the performance expected for isolated and fixed-base structures designed in accordance with the standard.

Loss of function is not included in Table C17.2-1. For certain fixed-base facilities, loss of function would not be expected unless there is significant structural damage causing closure or restricted access to the building. In other cases, a facility with only limited or no structural damage would not be functional as a result of damage to vital nonstructural components or contents. Isolation would be expected to mitigate structural and nonstructural damage and to protect the facility against loss of function.

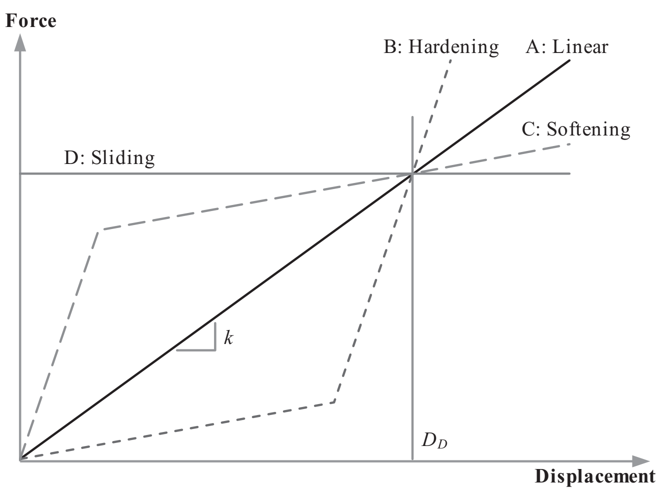

FIGURE C17.1-1 Idealized Force-Deflection Relationships for Isolation Systems (Stiffness Effects of Sacrificial Wind-Restraint Systems not Shown for Clarity)

C17.2.4 Isolation System

C17.2.4.1 Environmental Conditions. Environmental conditions that may adversely affect isolation system performance must be investigated thoroughly. Related research has been conducted since the 1970s in Europe, Japan, New Zealand, and the United States.

C17.2.4.2 Wind Forces. Lateral displacement over the depth of the isolator zone resulting from wind loads must be limited to a value similar to that required for other story heights.

C17.2.4.3 Fire Resistance. Where fire may adversely affect the lateral performance of the isolation system, the system must be protected so as to maintain the gravity-load resistance required for the other elements of the structure supported by the isolation system.

C17.2.4.4 Lateral Restoring Force. The restoring-force requirement is intended to limit residual displacement as a result of an earthquake so that the isolated structure survives aftershocks and future earthquakes.

C17.2.4.5 Displacement Restraint. The use of a displacement restraint is discouraged. Where a displacement restraint system is used, explicit analysis of the isolated structure for maximum considered earthquake (MCE) response is required to account for the effects of engaging the displacement restraint.

C17.2.4.6 Vertical-Load Stability. The vertical loads to be used in checking the stability of any given isolator should be calculated using bounding values of dead load and live load and the peak earthquake demand of the MCE. Because earthquake loads are reversible in nature, peak earthquake load should be combined with bounding values of dead and live load in a manner that produces both the maximum downward force and the maximum upward force on any isolator. Stability of each isolator should be verified for these two extreme values of vertical load at peak MCE displacement of the isolation system. In addition, all elements of the isolation system need testing or equivalent actions that demonstrate that they are stable for the MCE_R ground motion levels. This stability can be demonstrated by performing a nonlinear static analysis for an MCE_R response displacement of the entire structural system, including the isolation system, and showing that stability is maintained. Alternatively, this stability can be demonstrated by performing a nonlinear dynamic analysis for the MCE_R motions using the same inelastic reductions for the design earthquake and acceptable capacities except that member and connection strengths can be taken as their nominal strengths with phi factors taken as 1.0.

C17.2.4.7 Overturning. The intent of this requirement is to prevent both global structural overturning and overstress of elements caused by local uplift. Isolator uplift is acceptable so long as the isolation system does not disengage from its horizontal-resisting connection detail. The connection details used in some

Table C17.2-1 Performance Expected for Minor, Moderate, and Major Earthquakes[a]

Performance Measure	Earthquake Ground Motion Level: Minor	Moderate	Major
Life-safety: Loss of life or serious injury is not expected	F, I	F, I	F, I
Structural damage: Significant structural damage is not expected	F, I	F, I	I
Nonstructural damage: Significant nonstructural or contents damage is not expected	F, I	I	I

[a]F indicates fixed base; I indicates isolated.

isolation systems are such that tension is not permitted on the system. Where the tension capacity of an isolator is used to resist uplift forces, design and testing in accordance with Sections 17.2.4.6 and 17.8.2.5 must be performed to demonstrate the adequacy of the system to resist tension forces at the total maximum displacement.

C17.2.4.8 Inspection and Replacement. Although most isolation systems do not need to be replaced after an earthquake, access for inspection and replacement must be provided and periodic inspection is required. After an earthquake, the isolation system should be inspected and any damaged elements replaced or repaired.

C17.2.4.9 Quality Control. A testing and inspection program is necessary for both fabrication and installation of the isolator units. Because seismic isolation is a rapidly evolving technology, it may be difficult to reference standards for testing and inspection. Reference can be made to standards for some materials, such as elastomeric bearings (ASTM D 4014). Similar standards are yet to be developed for other isolation systems. Special inspection procedures and load testing to verify manufacturing quality must be developed for each project. The requirements vary depending on the type of isolation system used.

C17.2.5 Structural System

C17.2.5.2 Building Separations. A minimum separation between the isolated structure and rigid obstructions is required to allow free movement of the superstructure in all lateral directions during an earthquake.

C17.2.6 Elements of Structures and Nonstructural Components. To accommodate the differential movement between the isolated building and the ground, flexible utility connections are required. In addition, stiff elements crossing the isolation interface (such as stairs, elevator shafts, and walls) must be detailed to accommodate the total maximum displacement without compromising life-safety.

C17.3 GROUND MOTION FOR ISOLATED SYSTEMS

C17.3.1 Design Spectra. Seismically isolated structures located on Site Class F sites and on sites with $S_1 \geq 0.6$ must be analyzed using response history analysis. For those cases, the response spectra must be site specific to account, in the analysis, for near-fault effects and for soft soil conditions, both of which are known to be important in the assessment of displacement demands in seismically isolated structures.

C17.3.2 Ground Motion Histories. The selection and scaling of ground motions for response history analysis requires fitting to the response spectra in the period range of $0.5T_D$ to $1.25T_M$, a range that is different from that for conventional structures $0.2T$ to $1.5T$. The following sections provide background on the two period ranges:

1. Period Range—Isolated Structures. The effective (fundamental) period of an isolated structure is based on amplitude-dependent, nonlinear (pushover) stiffness properties of the isolation system. The effective periods, T_D and T_M, correspond to design earthquake displacement and MCE displacement, respectively, in the direction under consideration. Values of effective (fundamental) periods, T_D and T_M, are typically in the range of 2 to 4 s, and the value of the effective period, T_D, typically is 15 to 25% less than the corresponding value of effective period, T_M.

   The response of an isolated structure is dominated by the fundamental mode in the direction of interest. The specified period range, $0.5T_D$ to $1.25T_M$, conservatively bounds amplitude-dependent values of the effective (fundamental) period of the isolated structure in the direction of interest, considering that individual earthquake records can affect response at effective periods somewhat longer than T_M or significantly shorter than T_D.
2. Period Range—Conventional, Fixed-Base Structures. The fundamental period, T, of a conventional, fixed-base structure is based on amplitude-independent, linear-elastic stiffness properties of the structure. In general, response of conventional, fixed-base structures is influenced by both the fundamental mode and higher modes in the direction under consideration. The period range, $0.2T$ to $1.5T$, is intended to bound the fundamental period, considering some period lengthening caused by nonlinear response of the structure (that is, inelastic periods up to $1.5T$) and periods corresponding to the more significant higher modes (that is, second and possibly third modes in the direction of interest).

C17.4 ANALYSIS PROCEDURE SELECTION

Three different analysis procedures are available for determining design-level seismic loads: the equivalent lateral force procedure, the response spectrum procedure, and the response history procedure. For the first procedure, simple, lateral-force formulas (similar to those for conventional, fixed-base structures) are used to determine peak lateral displacement and design forces as a function of spectral acceleration and isolated-structure period and damping. For the second and third procedures, which are required for geometrically complex or especially flexible buildings, dynamic analysis (either the response spectrum procedure or the response history procedure) is used to determine peak response of the isolated structure.

The three procedures are based on the same level of seismic input and require a similar level of performance from the building. Where more complex analysis procedures are used, slightly lower design forces and displacements are permitted. The design requirements for the structural system are based on the design earthquake, taken as two-thirds of the MCE. The isolation system—including all connections, supporting structural elements, and the "gap"—is required to be designed (and tested) for 100% of MCE demand. Structural elements above the isolation system are not required to be designed for the full effects of the design earthquake but may be designed for slightly reduced loads (that is, loads reduced by a factor of up to 2) if the structural system is able to respond inelastically without sustaining significant damage. A similar fixed-base structure would be designed for loads reduced by a factor of 8 rather than 2.

This section delineates the requirements for the use of the equivalent lateral force procedure and dynamic methods of analysis. The limitations on the simplified lateral-force design procedure are quite restrictive. Limitations relate to the site location with respect to major, active faults; soil conditions of the site; the height, regularity, and stiffness characteristics of the building; and selected characteristics of the isolation system. Response history analysis is required to determine the design displacement of the isolation system (and the structure above) for the following isolated structures:

1. Isolated structures with a "nonlinear" isolation system including, but not limited to, isolation systems with effective damping values greater than 30% of critical, isolation systems incapable of producing a significant restoring

Minimum Design Loads for Buildings and Other Structures

Table C17.4-1 Lower-Bound Limits on Dynamic Procedures Specified in Relation to ELF Procedure Requirements

Design Parameter	ELF Procedure	Dynamic Procedure: Response Spectrum	Dynamic Procedure: Response History
Design displacement: D_D	$D_D = (g/4\pi^2)(S_{D1}T_D/B_D)$	—	—
Total design displacement: D_{TD}	$D_{TD} \geq 1.1D_D$	$\geq 0.9D_{TD}$	$\geq 0.9D_{TD}$
Maximum displacement: D_M	$D_M = (g/4\pi^2)(S_{M1}T_M/B_M)$	—	—
Total maximum displacement: D_{TM}	$D_{TM} \geq 1.1D_M$	$\geq 0.8D_{TM}$	$\geq 0.8D_{TM}$
Design shear: V_b (at or below the isolation system)	$V_b = k_{D\max}D_D$	$\geq 0.9V_b$	$\geq 0.9V_b$
Design shear: V_s ("regular" superstructure)	$V_s = k_{D\max}D_D/R_I$	$\geq 0.8V_s$	$\geq 0.6V_s$
Design shear: V_s ("irregular" superstructure)	$V_s = k_{D\max}D_D/R_I$	$\geq 1.0V_s$	$\geq 0.8V_s$
Drift (calculated using R_I for C_d)	$0.015h_{sx}$	$0.015h_{sx}$	$0.020h_{sx}$

force, and isolation systems that restrain or limit extreme earthquake displacement; and

2. Isolated structures located on a Class E or Class F site (that is, a soft-soil site that amplifies long-period ground motions).

Lower-bound limits on isolation system design displacements and structural-design forces are specified by the standard in Section 17.6 as a percentage of the values prescribed by the equivalent lateral force procedure, even where dynamic analysis is used as the basis for design. These lower-bound limits on key design parameters provide consistency in the design of isolated structures and serve as a safety net against gross underdesign. Table C17.4-1 provides a summary of the lower-bound limits on dynamic analysis specified by the standard.

C17.5 EQUIVALENT LATERAL FORCE PROCEDURE

C17.5.3 Minimum Lateral Displacements. The lateral displacement given by Eq. 17.5-1 approximates peak design earthquake displacement of a single-degree-of-freedom, linear-elastic system of period, T_D, and effective damping, β_D. Similarly, the lateral displacement given by Eq. 17.5-3 approximates peak MCE displacement of a single-degree-of-freedom, linear-elastic system of period, T_M, and effective damping, β_M.

Equation 17.5-1 is an estimate of peak displacement in the isolation system for the design earthquake. In this equation, the spectral acceleration term, S_{D1}, is the same as that required for design of a conventional, fixed-base structure of period, T_D. A damping term, B_D, is used to decrease (or increase) the computed displacement where the effective damping coefficient of the isolation system is greater (or smaller) than 5% of critical damping. Values of coefficient B_D (or B_M for the MCE) are given in Table 17.5-1 for different values of isolation system damping, β_D (or β_M).

A comparison of values obtained from Eq. 17.5-1 and those obtained from nonlinear time history analyses are given in Kircher et al. (1988) and Constantinou et al. (1993).

Consideration should be given to possible differences in the properties used for design of the isolation system and those of the isolation system as installed in the structure. Similarly, consideration should be given to possible changes in isolation system properties caused by different design conditions or load combinations. If the true deformational characteristics of the isolation system are not stable or if they vary with the nature of the load (being rate-, amplitude-, or time-dependent), the design displacements should be based on deformational characteristics of the isolation system that give the largest possible deflection ($k_{D\min}$), the design forces should be based on deformational

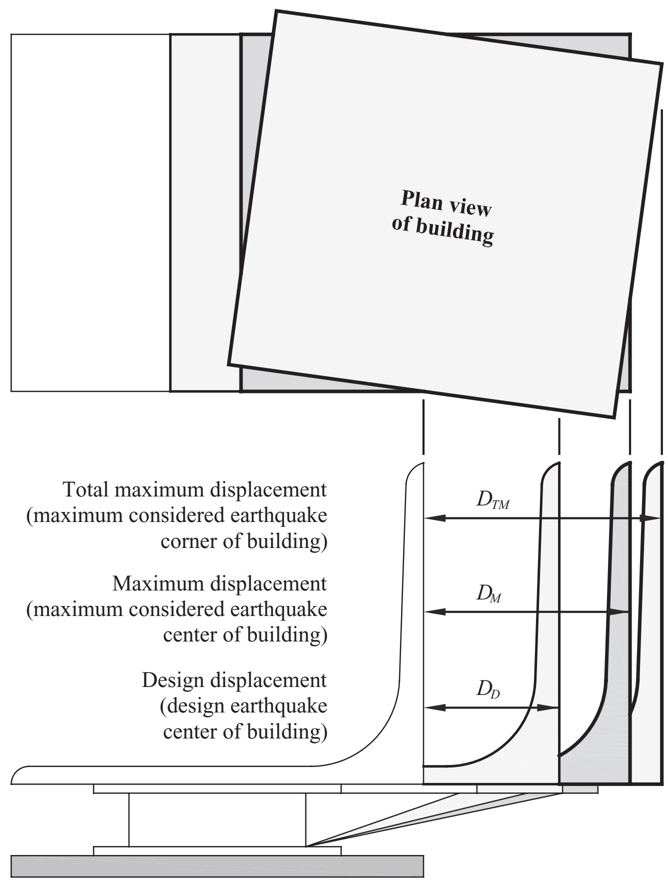

FIGURE C17.5-1 Displacement Terminology

characteristics of the isolation system that give the largest possible force ($k_{D\max}$), and the damping level used to determine design displacements and forces should be based on deformational characteristics of the isolation system that represent the minimum amount of energy dissipated during cyclic response at the design level.

The isolation system for a seismically isolated structure should be configured to minimize eccentricity between the center of mass of the superstructure and the center of rigidity of the isolation system. In this way, the effect of torsion on the displacement of isolation elements is reduced. As for conventional structures, allowance must be made for accidental eccentricity in both horizontal directions. Figure C17.5-1 illustrates the terminology used in the standard. Equation 17.5-5 (or Eq. 17.5-6 for the

MCE) provides a simplified formula for estimating the response caused by torsion in lieu of a more refined analysis. The additional component of displacement caused by torsion increases the design displacement at the corner of a structure by about 15% (for one perfectly square in plan) to about 30% (for one very long and rectangular in plan) if the eccentricity is 5% of the maximum plan dimension. These calculated torsional displacements are for structures with an isolation system whose stiffness is uniformly distributed in plan. Isolation systems that have stiffness concentrated toward the perimeter of the structure, or certain sliding systems that minimize the effects of mass eccentricity, have smaller torsional displacements. The standard permits values of D_{TD} as small as $1.1D_D$, with proper justification.

C17.5.4 Minimum Lateral Forces. Figure C17.5-2 illustrates the terminology for elements at, below, and above the isolation system. Equation 17.5-7 specifies the peak seismic shear for design of all structural elements at or below the isolation system (without reduction for ductile response). Equation 17.5-8 specifies the peak seismic shear for design of structural elements above the isolation system. For structures that have appreciable inelastic-deformation capability, this equation includes an effective reduction factor of up to 2 for response beyond the strength-design level.

The reduction factor is based on use of strength-design procedures. A factor of at least 2 is assumed to exist between the design-force level and the true-yield level of the structural system. An investigation of 10 specific buildings indicated that this factor varied between 2 and 5 (ATC 1982). Thus, a reduction factor of 2 is appropriate to produce a structural system that remains essentially elastic for the design earthquake.

In Section 17.5.4.3, the limits given on V_s provide at least a factor of 1.5 between the nominal yield level of the superstructure and (a) the yield level of the isolation system, (b) the ultimate capacity of a sacrificial wind-restraint system that is intended to fail and release the superstructure during significant lateral load, or (c) the breakaway friction level of a sliding system.

These limits are needed so that the superstructure does not yield prematurely before the isolation system has been activated and significantly displaced.

The design shear force, V_s, specified in this section results in an isolated structural system being subjected to significantly lower inelastic demands than a conventionally designed structural system. Further reduction in V_s, such that the inelastic demand on a seismically isolated structure would be the same as the inelastic demand on a conventionally designed structure, was not considered during development of these requirements but may be considered in the future.

Using a smaller value of R_I in Eq. 17.5-8 reduces or eliminates inelastic response of the superstructure for the design-basis event, thus further improving the structural performance.

C17.5.5 Vertical Distribution of Force. Equation 17.5-9 produces a vertical distribution of lateral forces consistent with a triangular profile of seismic acceleration over the height of the structure above the isolation interface. Kircher et al. (1988) and Constantinou et al. (1993) show that Eq. 17.5-9 provides a conservative estimate of the distributions obtained from more detailed, nonlinear analysis studies for the type of structures for which use of Eq. 17.5-9 is allowed.

C17.5.6 Drift Limits. The maximum story drift permitted for design of isolated structures is constant for all risk categories, as shown in Table C17.5-1. For comparison, the drift limits prescribed by the standard for fixed-base structures also are summarized in that table.

Drift limits in Table C17.5-1 are divided by C_d/R for fixed-base structures because displacements calculated for lateral loads reduced by R are multiplied by C_d before checking drift. The C_d term is used throughout the standard for fixed-base structures to approximate the ratio of actual earthquake response to response calculated for reduced forces. Generally, C_d is 1/2 to

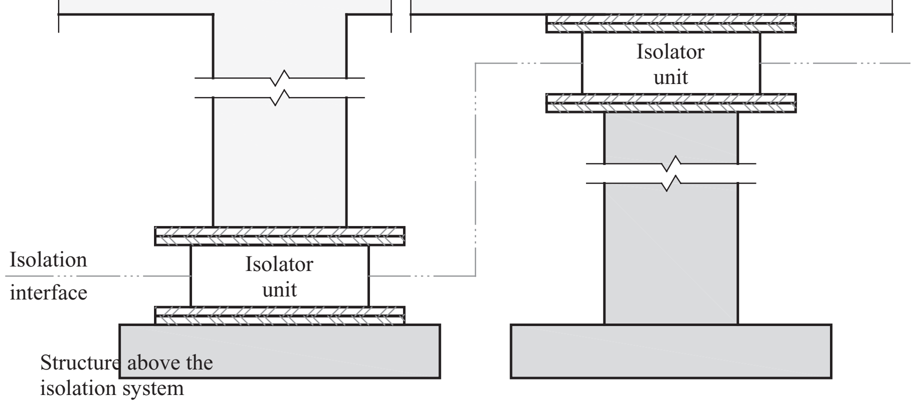

FIGURE C17.5-2 Isolation System Terminology

Table C17.5-1 Comparison of Drift Limits for Fixed-Base and Isolated Structures

Structure	Risk Category	Fixed-Base	Isolated
Buildings (other than masonry) four stories or fewer in height with component drift design	I or II	$0.025h_{sx}/(C_d/R)$	$0.015h_{sx}$
	III	$0.020h_{sx}/(C_d/R)$	$0.015h_{sx}$
	IV	$0.015h_{sx}/(C_d/R)$	$0.015h_{sx}$
Other (nonmasonry) buildings	I or II	$0.020h_{sx}/(C_d/R)$	$0.015h_{sx}$
	III	$0.015h_{sx}/(C_d/R)$	$0.015h_{sx}$
	IV	$0.010h_{sx}/(C_d/R)$	$0.015h_{sx}$

4/5 the value of R. For isolated structures, the R_I factor is used both to reduce lateral loads and to increase displacements (calculated for reduced lateral loads) before checking drift. Equivalency would be obtained if the drift limits for both fixed-base and isolated structures were based on their respective R factors. It may be noted that the drift limits for isolated structures generally are more conservative than those for conventional, fixed-base structures, even where fixed-base structures are assigned to Risk Category IV.

C17.6 DYNAMIC ANALYSIS PROCEDURES

This section specifies the requirements and limits for dynamic procedures. The design displacement and force limits on response spectrum and response history procedures are shown in Table C17.4-1.

A more detailed or refined study can be performed in accordance with the analysis procedures described in this section. The intent of this section is to provide procedures that are compatible with the minimum requirements of Section 17.5. Reasons for performing a more refined study include:

1. The importance of the building.
2. The need to analyze possible structure-isolation system interaction where the fixed-base period of the building is greater than one-third of the isolated period.
3. The need to explicitly model the deformational characteristics of the lateral force-resisting system where the structure above the isolation system is irregular.
4. The desirability of using site-specific ground motion data, especially for very soft or liquefiable soils (Site Class F) or for structures located where S_1 is greater than 0.60.
5. The desirability of explicitly modeling the deformational characteristics of the isolation system. This point is especially important for systems that have damping characteristics that are amplitude-dependent, rather than velocity-dependent, because it is difficult to determine an appropriate value of equivalent viscous damping for these systems.

Section C17.4 discusses other conditions that require use of the response history procedure. As shown in Table C17.4-1, the drift limit for isolated structures is relaxed where story drifts are computed using nonlinear response history analysis.

Where response history analysis is used as the basis for design, the design displacement of the isolation system and design forces in elements of the structure above are computed from not fewer than three separate analyses, each using a different ground motion selected and scaled in accordance with Section 17.3.2. Where the configuration of the isolation system or of the superstructure is not symmetric, additional analyses are required to satisfy the requirement of Section 17.6.3.4 to consider the most disadvantageous location of eccentric mass. As appropriate, near-field phenomena may also be incorporated. As in the nuclear industry, where at least seven ground motions are used for nonlinear response history analysis, it is considered appropriate to base design of seismically isolated structures on the average value of the response parameters of interest.

C17.7 DESIGN REVIEW

Review of the design and analysis of the isolation system and design review of the isolator testing program is mandated by the standard for two key reasons:

1. The consequences of isolator failure could be catastrophic.
2. Isolator design and fabrication technology is evolving rapidly and may be based on technologies unfamiliar to many design professionals.

The standard requires review to be performed by registered design professionals who are independent of the design team and other project contractors. The review team should include individuals with special expertise in one or more aspects of the design, analysis, and implementation of seismic isolation systems.

The review team should be formed before the development of design criteria (including site-specific ground shaking criteria) and isolation system design options. Furthermore, the review team should have full access to all pertinent information and the cooperation of the design team and regulatory agencies involved with the project.

C17.8 TESTING

The design displacements and forces determined using the standard assume that the deformational characteristics of the isolation system have been defined previously by comprehensive testing. If comprehensive test data are not available for a system, major design alterations in the structure may be necessary after the tests are complete. This would result from variations in the isolation system properties assumed for design and those obtained by test. Therefore, it is advisable that prototype systems be tested during the early phases of design, if sufficient test data are not available on an isolation system.

Typical force-deflection (or hysteresis) loops are shown in Fig. C17.8-1; also illustrated are the values defined in Section 17.8.5.1.

The required sequence of tests experimentally verifies the following:

1. The assumed stiffness and capacity of the wind-restraining mechanism;
2. The variation of the isolator's deformational characteristics with amplitude (and with vertical load, if it is a vertical load-carrying member);

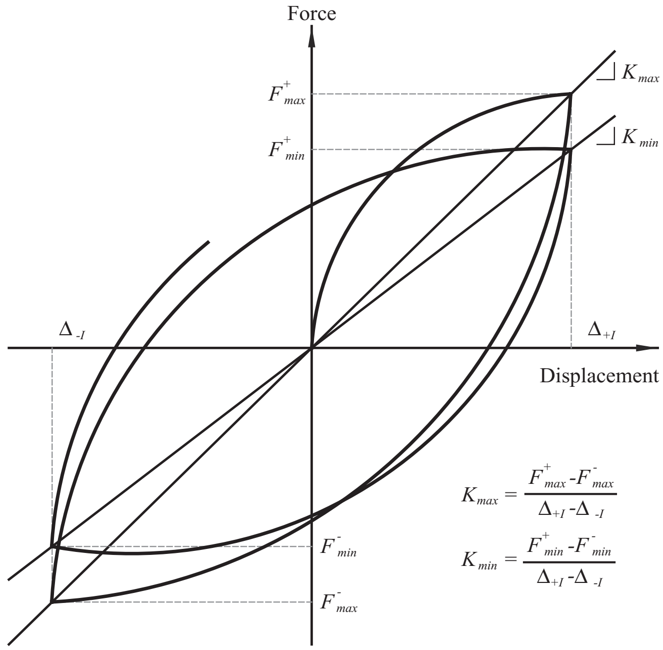

FIGURE C17.8-1 The Effect of Stiffness on an Isolation Bearing

3. The variation of the isolator's deformational characteristics for a realistic number of cycles of loading at the design displacement; and
4. The ability of the system to carry its maximum and minimum vertical loads at the maximum displacement.

Force-deflection tests are not required if similarly sized components have been tested previously using the specified sequence of tests.

The variations in the vertical loads required for tests of isolators that carry vertical and lateral loads are necessary to determine possible variations in the system properties with variations in overturning force.

C17.8.5 Design Properties of the Isolation System

C17.8.5.1 Maximum and Minimum Effective Stiffness. The effective stiffness is determined from the hysteresis loops, as shown in Fig. C17.8-1. Stiffness may vary considerably as the test amplitude increases but should be reasonably stable (within 15%) for more than three cycles at a given amplitude.

The intent of these requirements is that the deformational properties used in design result in the maximum design forces and displacements. For determining design displacement, this means using the smallest damping and effective stiffness values. For determining design forces, this means using the smallest damping value and the largest stiffness value.

C17.8.5.2 Effective Damping. The determination of effective damping is reasonably reliable for systems whose damping characteristics are velocity dependent. For systems that have amplitude-dependent energy-dissipating mechanisms, significant problems arise in determining an effective damping value. Because it is difficult to relate velocity- and amplitude-dependent phenomena, it is recommended that where the effective damping assumed for the design of amplitude-dependent energy-dissipating mechanisms (such as pure-sliding systems) is greater than 30%, the design-basis force and displacement be determined using the response history procedure, as discussed in Section C17.4.

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). (1999). *Guide Specification for Seismic Isolation Design*. AASHTO, Washington, D.C.

American Society of Civil Engineers (ASCE). (2007). "Seismic rehabilitation of existing buildings." *ASCE/SEI 41*, ASCE, Reston, Va.

Applied Technology Council. (1982). *An Investigation of the Correlation between Earthquake Ground Motion and Building Performance*, ATC Report 10. ATC, Redwood City, Calif.

ASTM International. ((2012). "Standard specification for plain and steel-laminated elastomeric bearings for bridges." *D4014*, ASTM.

Constantinou, M. C., Winters, C. W., and Theodossiou, D. (1993). "Evaluation of SEAOC and UBC analysis procedures, Part 2: Flexible superstructure." *Proceedings of a Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control*, ATC Report 17-1. ATC, Redwood City, Calif.

Constantinou, M. C., Tsopelas, P., Kasalanati, A., and Wolff, E. D. (1999). "Property modification factors for seismic isolation bearings." *MCEER-99-0012*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, N.Y.

Constantinou, M. C., Whittaker, A. S., Kalpakidis, Y., Fenz, D. M., and Warn, G. P. (2007). "Performance of seismic isolation hardware under service and seismic loading," *MCEER-07-0012*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, N.Y.

International Code Council. (2006). *International Building Code.* ICC, Washington, D.C.

Kircher, C. A., Lashkari, B., Mayes, R. L., and Kelly, T. E. (1988). "Evaluation of nonlinear response in seismically isolated buildings." *Proceedings of a Symposium on Seismic, Shock, and Vibration Isolation*, ASME PVP Conference.

Lashkari, B., and Kircher, C. A. (1993). "Evaluation of SEAOC & UBC analysis procedures, Part 1: stiff superstructure." *Proceedings of a Seminar on Seismic Isolation, Passive Energy Dissipation and Acti.*, ATC.

CHAPTER C18
SEISMIC DESIGN REQUIREMENTS FOR STRUCTURES WITH DAMPING SYSTEMS

C18.1 GENERAL

The requirements of this chapter apply to all types of damping systems, including both displacement-dependent damping devices of hysteretic or friction systems and velocity-dependent damping devices of viscous or viscoelastic systems (Soong and Dargush 1997, Constantinou et al. 1998, Hanson and Soong 2001). Compliance with these requirements is intended to produce performance comparable to that of a structure with a conventional seismic force-resisting system, but the same methods can be used to achieve higher performance.

The damping system (DS) is defined separately from the seismic force-resisting system (SFRS), although the two systems may have common elements. As illustrated in Fig. C18.1-1, the DS may be external or internal to the structure and may have no shared elements, some shared elements, or all elements in common with the SFRS. Elements common to the DS and the SFRS must be designed for a combination of the two loads of the two systems. When the DS and SFRS have no common elements, the damper forces must be collected and transferred to members of the SFRS.

C18.2 GENERAL DESIGN REQUIREMENTS

C18.2.2 System Requirements. Structures with a DS must have a SFRS that provides a complete load path. The SFRS must comply with all of the height, seismic design category, and redundancy limitations and with the detailing requirements specified in this standard for the specific SFRS. The SFRS without the damping system (as if damping devices were disconnected) must be designed to have not less than 75% of the strength required for undamped structures that have that type of SFRS (and not less than 100% if the structure is horizontally or vertically irregular). The damping systems, however, may be used to meet the drift limits (whether the structure is regular or irregular). Having the SFRS designed for a minimum of 75% of the strength required for undamped structures provides safety in the event of damping system malfunction and produces a composite system with sufficient stiffness and strength to have controlled lateral displacement response.

The DS must be designed for the actual (unreduced) earthquake forces (such as peak force occurring in damping devices) and deflections. For certain elements of the DS (such as the connections or the members into which the damping devices frame), other than damping devices, limited yielding is permitted provided such behavior does not affect damping system function or exceed the amount permitted for elements of conventional structures by the standard.

C18.2.4 Procedure Selection. Linear static and response spectrum analysis methods can be used for design of structures with damping systems that meet certain configuration and other limiting criteria (for example, at least two damping devices at each story configured to resist torsion). In such cases, additional nonlinear response history analysis shall be used to confirm peak responses when the structure is located at a site with S_1 greater than or equal to 0.6. The analysis methods of damped structures are based on nonlinear static "pushover" characterization of the structure and calculation of peak response using effective (secant) stiffness and effective damping properties of the first (pushover) mode in the direction of interest. These concepts are used in Chapter 17 to characterize the force-deflection properties of isolation systems, modified to incorporate explicitly the effects of ductility demand (postyield response) and higher mode response of structures with dampers. Like conventional structures, damped structures generally yield during strong ground shaking, and their performance can be influenced strongly by response of higher modes.

The response spectrum and equivalent lateral force procedures presented in the standard have several simplifications and limits, as outlined below:

1. A multiple-degree-of-freedom (MDOF) structure with a damping system can be transformed into equivalent single-degree-of-freedom (SDOF) systems using modal decomposition procedures. This transformation assumes that the collapse mechanism for the structure is a SDOF mechanism so that the drift distribution over height can be estimated reasonably using either the first mode shape or another profile, such as an inverted triangle. Such procedures do not strictly apply to either yielding buildings or buildings that are nonproportionally damped.
2. The response of an inelastic SDOF system can be estimated using equivalent linear properties and a 5%-damped response spectrum. Spectra for damping greater than 5% can be established using damping coefficients, and velocity-dependent forces can be established either by using the pseudovelocity and modal information or by applying correction factors to the pseudovelocity.
3. The nonlinear response of the structure can be represented by a bilinear hysteretic relationship with zero postelastic stiffness (elastoplastic behavior).
4. The yield strength of the structure can be estimated either by performing simple plastic analysis or by using the specified minimum seismic base shear and values of R, Ω_0, and C_d.
5. Higher modes need to be considered in the equivalent lateral force procedure to capture their effects on velocity-dependent forces. This is reflected in the residual mode procedure.

FEMA 440 (2005) presents a review of simplified procedures for the analysis of yielding structures. The combined effects of the

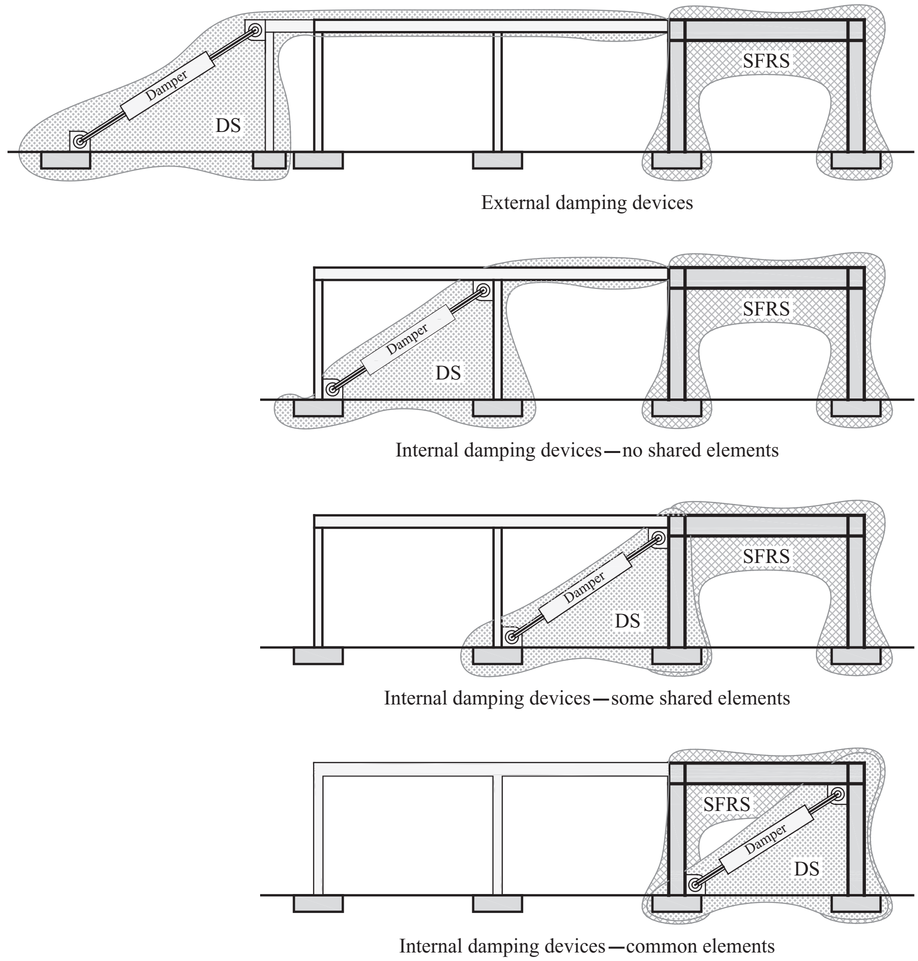

FIGURE C18.1-1 Damping System (DS) and Seismic Force-Resisting System (SFRS) Configurations

simplifications mentioned previously are reported by Ramirez et al. (2001) and Pavlou and Constantinou (2004) based on studies of three-story and six-story buildings with damping systems designed by the procedures of the standard. The response spectrum and equivalent lateral force procedures of the standard are found to provide conservative predictions of drift and predictions of damper forces and member actions that are of acceptable accuracy when compared with results of nonlinear dynamic response history analysis. When designed in accordance with the standard, structures with damping systems are expected to have structural performance at least as good as that of structures without damping systems. Pavlou and Constantinou (2006) report that structures with damping systems designed in accordance with the standard provide the benefit of reduced secondary system response, although this benefit is restricted to systems with added viscous damping.

C18.3 NONLINEAR PROCEDURES

For designs in which the seismic force-resisting-system is essentially elastic (assuming an overstrength of 50%), the only nonlinear characteristics that must be modeled in the analysis are those of the damping devices. For designs in which the seismic force-resisting system yields, the postyield behavior of the structural elements must be modeled explicitly.

C18.4 RESPONSE-SPECTRUM PROCEDURE and C18.5 EQUIVALENT LATERAL FORCE PROCEDURE

Effective Damping. In the standard, the reduced response of a structure with a damping system is characterized by the damping coefficient, B, based on the effective damping, β, of the mode of interest. This approach is the same as that used for isolated structures. Like isolation, effective damping of the fundamental mode of a damped structure is based on the nonlinear force-deflection properties of the structure. For use with linear analysis methods, nonlinear properties of the structure are inferred from the overstrength factor, Ω_0, and other terms.

Figure C18.4-1 illustrates reduction in design earthquake response of the fundamental mode caused by increased effective damping (represented by coefficient, B_{1D}). The capacity curve is a plot of the nonlinear behavior of the fundamental mode in spectral acceleration-displacement coordinates. The reduction caused by damping is applied at the effective period of the fundamental mode of vibration (based on the secant stiffness).

In general, effective damping is a combination of three components:

1. Inherent Damping (β_I)—Inherent damping of the structure at or just below yield, excluding added viscous damping

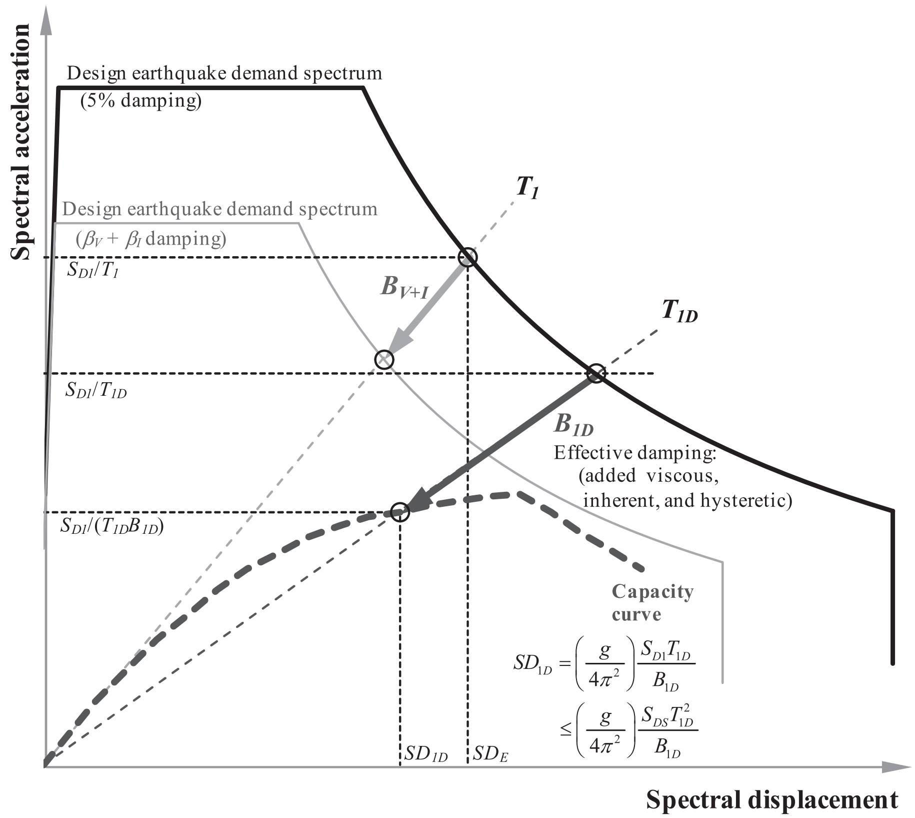

FIGURE C18.4-1 Effective Damping Reduction of Design Demand

(typically assumed to be 5% of critical for structural systems without dampers).

2. Hysteretic Damping (β_H)—Postyield hysteretic damping of the seismic force-resisting system and elements of the damping system at the amplitude of interest (taken as 0% of critical at or below yield).
3. Added Viscous Damping (β_V)—The viscous component of the damping system (taken as 0% for hysteretic or friction-based damping systems).

Both hysteretic damping and added viscous damping are amplitude-dependent, and the relative contributions to total effective damping change with the amount of postyield response of the structure. For example, adding dampers to a structure decreases postyield displacement of the structure and, hence, decreases the amount of hysteretic damping provided by the seismic force-resisting system. If the displacements are reduced to the point of yield, the hysteretic component of effective damping is zero, and the effective damping is equal to inherent damping plus added viscous damping. If there is no damping system (as in a conventional structure), effective damping simply equals inherent damping (typically assumed to be 5% of critical for most conventional structures).

Linear Analysis Methods. The section specifies design earthquake displacements, velocities, and forces in terms of design earthquake spectral acceleration and modal properties. For equivalent lateral force (ELF) analysis, response is defined by two modes: the fundamental mode and the residual mode. The residual mode is a new concept used to approximate the combined effects of higher modes. Although typically of secondary importance to story drift, higher modes can be a significant contributor to story velocity and, hence, are important for design of velocity-dependent damping devices. For response spectrum analysis, higher modes are explicitly evaluated.

For both the ELF and the response spectrum analysis procedures, response in the fundamental mode in the direction of interest is based on assumed nonlinear (pushover) properties of the structure. Nonlinear (pushover) properties, expressed in terms of base shear and roof displacement, are related to building capacity, expressed in terms of spectral coordinates, using mass participation and other fundamental-mode factors shown in Fig. C18.4-2. The conversion concepts and factors shown in Fig. C18.4-2 are the same as those defined in Chapter 9 of ASCE/SEI 41 (2007), which addresses seismic rehabilitation of a structure with damping devices.

Where using linear analysis methods, the shape of the fundamental-mode pushover curve is not known, so an idealized elastoplastic shape is assumed, as shown in Fig. C18.4-3. The idealized pushover curve is intended to share a common point with the actual pushover curve at the design earthquake displacement, D_{1D}. The idealized curve permits definition of the global ductility demand caused by the design earthquake, μ_D, as the ratio of design displacement, D_{1D}, to yield displacement, D_Y. This ductility factor is used to calculate various design factors; it must not exceed the ductility capacity of the seismic force-resisting system, μ_{max}, which is calculated using factors for conventional structural response. Design examples using linear analysis methods have been developed and found to compare well with the results of nonlinear time history analysis (Ramirez et al. 2001).

Elements of the damping system are designed for fundamental-mode design earthquake forces corresponding to a base shear value of V_Y (except that damping devices are designed and

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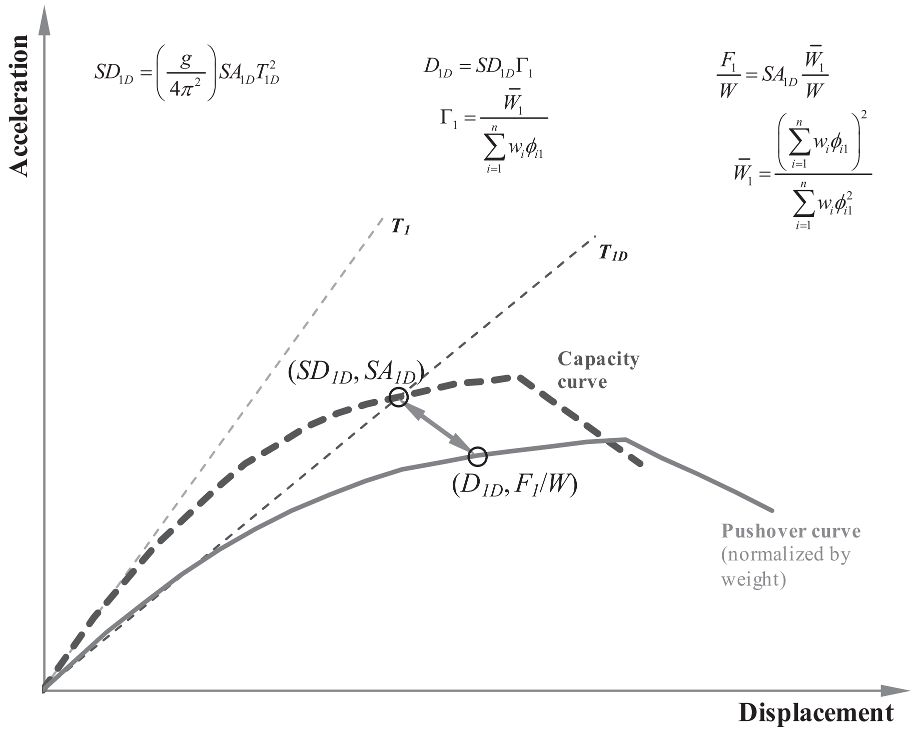

FIGURE C18.4-2 Pushover and capacity curves

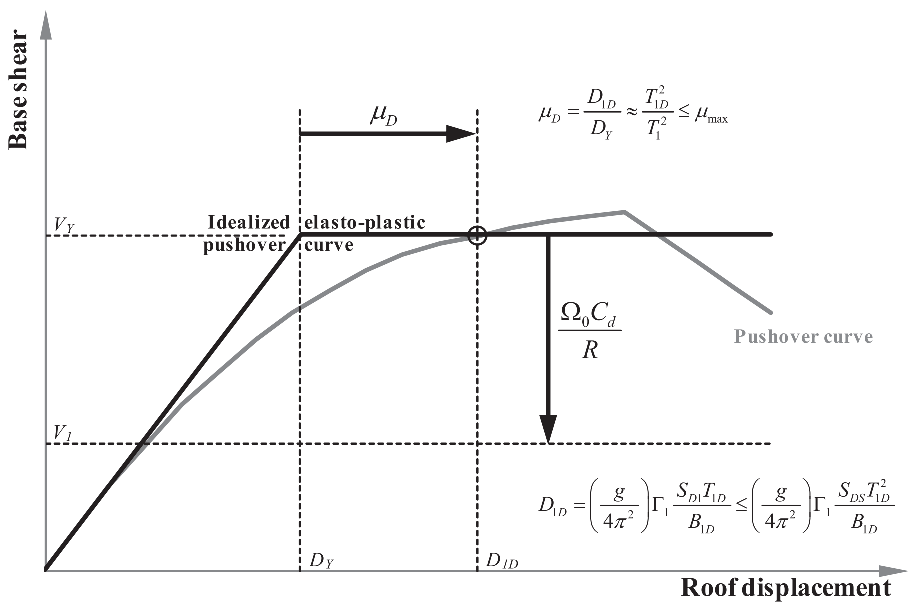

FIGURE C18.4-3 Pushover and capacity curves

prototypes are tested for maximum considered earthquake response). Elements of the seismic force-resisting system are designed for reduced fundamental-mode base shear, V_1, where force reduction is based on system overstrength (represented by Ω_0), multiplied by C_d/R for elastic analysis (where actual pushover strength is not known). Reduction using the ratio C_d/R is necessary because the standard provides values of C_d that are less than those for R. Where the two parameters have equal value and the structure is 5% damped under elastic conditions, no adjustment is necessary. Because the analysis methodology is based on calculating the actual story drifts and damping device displacements (rather than the displacements calculated for elastic conditions at the reduced base shear and then multiplied by C_d), an adjustment is needed. Because actual story drifts are calculated, the allowable story drift limits of Table 12.12-1 are multiplied by R/C_d before use.

C18.6 DAMPED RESPONSE MODIFICATION

C18.6.1 Damping Coefficient. Values of the damping coefficient, B, in Table 18.6-1 for design of damped structures are the same as those in Table 17.5-1 for isolated structures at damping

Table C18.6-1 Values of Damping Coefficient, B

Effective Damping, β(%)	Table 17.5-1 of ASCE/SEI (2010), AASHTO (1999), CBC (2001, seismically isolated structures)	Table 18.6-1 of ASCE/SEI (2010) (structures with damping systems)	FEMA 440 (2005)	Eurocode 8 (2005)
2	0.8	0.8	0.8	0.8
5	1.0	1.0	1.0	1.0
10	1.2	1.2	1.2	1.2
20	1.5	1.5	1.5	1.6
30	1.7	1.8	1.8	1.9
40	1.9	2.1	2.1	2.1
50	2.0	2.4	2.4	2.3

levels up to 20%, but extend to higher damping levels based on results presented in Ramirez et al. (2001). Table C18.6-1 compares values of the damping coefficient as found in the standard and various resource documents and codes. FEMA 440 and the draft of Eurocode 8 present equations for the damping coefficient, B, whereas the other documents present values of B in tabular format.

The equation in FEMA 440 is

$$B = \frac{4}{5.6 - \ln(100\beta)}$$

The equation in Eurocode 8 (2005) is

$$B = \sqrt{\frac{0.05 + \beta}{0.10}}$$

C18.6.2 Effective Damping. The effective damping is calculated assuming the structural system exhibits perfectly bilinear hysteretic behavior characterized by the effective ductility demand, μ, as described in Ramirez et al. (2001). Effective damping is adjusted using the hysteresis loop adjustment factor, q_H, which is the actual area of the hysteresis loop divided by the area of the assumed perfectly bilinear hysteretic loop. In general, values of this factor are less than unity. In Ramirez et al. (2001), expressions for this factor (which they call Quality Factor) are too complex to serve as a simple rule. Equation 18.6-5 provides a simple estimate of this factor. The equation predicts correctly the trend in the constant acceleration domain of the response spectrum, and it is believed to be conservative for flexible structures.

C18.7 SEISMIC LOAD CONDITIONS AND ACCEPTANCE CRITERIA

C18.7.2.5 Seismic Load Conditions and Combination of Modal Responses. Seismic design forces in elements of the damping system are calculated at three distinct stages: maximum displacement, maximum velocity, and maximum acceleration. All three stages need to be checked for structures with velocity-dependent damping systems. For displacement-dependent damping systems, the first and third stages are identical, whereas the second stage is inconsequential.

Force coefficients C_{mFD} and C_{mFV} are used to combine the effects of forces calculated at the stages of maximum displacement and maximum velocity to obtain the forces at maximum acceleration. The coefficients are presented in tabular form based on analytic expressions presented in Ramirez et al. (2001) and account for nonlinear viscous behavior and inelastic structural system behavior.

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). (1999). *Guide Specifications for Seismic Isolation Design*, Washington, D.C.

American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI). (2007). "Seismic rehabilitation of existing buildings." *ASCE/SEI 41-06*, ASCE, Reston, Va.

California Buildings Standards Commission (CBC). (2007). *California Building Code*. Sacramento, Calif.

Constantinou, M. C., Soong, T. T., and Dargush, G. F. (1998). *Passive Energy Dissipation Systems for Structural Design and Retrofit*, Monograph 1, Multidisciplinary Center for Earthquake Engineering Research, University of Buffalo, State University of New York, Buffalo, N.Y.

European Committee for Standardization (Eurocode 8). (2005). "Design of structures for earthquake resistance. Part 2: Bridges." *Eurocode 8*, European Committee for Standardization.

Federal Emergency Management Agency (FEMA). (2005). "Improvement of nonlinear static seismic analysis procedures." *FEMA 440*, Federal Emergency Management Agency, Washington, D.C.

Hanson, R. D., and Soong, T. T. (2001). "Seismic design with supplemental energy dissipation devices." *MNO-8*, Earthquake Engineering Research Institute, Oakland, Calif.

Pavlou, E., and Constantinou, M. C. (2004). "Response of elastic and inelastic structures with damping systems to near-field and soft-soil ground motions." *Engrg. Struct.*, 26, 1217–1230.

Pavlou, E., and Constantinou, M. C. (2006). "Response of nonstructural components in structures with damping systems." *J. Struct. Engrg.*, 132(7), 1108–1117.

Ramirez, O. M., Constantinou, M. C., Kircher, C. A., Whittaker, A., Johnson, M., Gomez, J. D., and Chrysostomou, C. Z. (2001). *Development and Evaluation of Simplified Procedures of Analysis and Design for Structures with Passive Energy Dissipation Systems*, Technical Report MCEER-00-0010, Revision 1, Multidisciplinary Center for Earthquake Engineering Research, University of Buffalo, State University of New York, Buffalo, N.Y.

Soong, T. T., and Dargush, G. F. (1997). *Passive Energy Dissipation Systems in Structural Engineering*, J. Wiley & Sons, London, U.K.

OTHER REFERENCES (NOT CITED)

Newmark, N. M., and Hall, W. J. (1969). "Seismic design criteria for nuclear reactor facilities." *Proceedings of the 4th World Conference in Earthquake Engineering*, Santiago, Chile.

Ramirez, O. M., Constantinou, M. C., Gomez, J., Whittaker, A. S., and Chrysostomou, C. Z. (2002a). "Evaluation of simplified methods of analysis of yielding structures with damping systems." *Earthquake Spectra*, 18(3), 501–530.

Ramirez, O. M., Constantinou, M. C., Whittaker, A. S., Kircher, C. A., and Chrysostomou, C. Z. (2002b). "Elastic and inelastic seismic response of buildings with damping systems." *Earthquake Spectra*, 18(3), 531–547.

Ramirez, O. M., Constantinou, M. C., Whittaker, A. S., Kircher, C. A., Johnson, M. W., and Chrysostomou, C. Z. (2003). "Validation of 2000 NEHRP provisions equivalent lateral force and modal analysis procedures for buildings with damping systems." *Earthquake Spectra*, 19(4), 981–999.

Whittaker, A. S., Constantinou, M. C., Ramirez, O. M., Johnson, M. W., and Chrysostomou, C. Z. (2003). "Equivalent lateral force and modal analysis procedures of the 2000 NEHRP provisions for buildings with damping systems." *Earthquake Spectra*, 19(4), 959–980.

Minimum Design Loads for Buildings and Other Structures

CHAPTER C19
SOIL-STRUCTURE INTERACTION FOR SEISMIC DESIGN

C19.1 GENERAL

The response of a structure to earthquake shaking is affected by interactions among three linked systems: the structure, the foundation, and the geologic media underlying and surrounding the foundation. A seismic soil-structure interaction (SSI) analysis evaluates the collective response of these systems to a specified free-field ground motion. The term "free-field" refers to motions not affected by structural vibrations and represents the condition for which the design spectrum is derived using the procedures given in Chapter 11.

SSI effects are absent for the theoretical condition of rigid foundation and soil conditions. Accordingly, SSI effects reflect the differences between the actual response of the structure and the response for the theoretical, rigid base condition. Visualized within this context, three SSI effects can significantly affect the response of building structures:

1. Foundation stiffness and damping. Inertia developed in a vibrating structure gives rise to base shear, moment, and torsional excitation, and these loads in turn cause displacements and rotations of the foundation relative to the free field. These relative displacements and rotations are only possible because of compliance in the soil-foundation system, which can significantly contribute to the overall structural flexibility in some cases. Moreover, the relative foundation free-field motions give rise to energy dissipation via radiation damping (i.e., damping associated with wave propagation into the ground away from the foundation, which acts as the wave source) and hysteretic soil damping, and this energy dissipation can significantly affect the overall damping of the soil-foundation-structure system. Because these effects are rooted in the structural inertia, they are referred to as inertial interaction effects.
2. Variations between free-field and foundation-level ground motions. The differences between foundation and free-field motions result from two processes. The first is known as kinematic interaction and results from the presence of stiff foundation elements on or in soil, which causes foundation motions to deviate from free-field motion as a result of base slab averaging, wave scattering, and embedment effects. Procedures for modifying design spectra to account for these effects are given in FEMA 440 and ASCE/SEI 41. The second process is related to the structure and foundation inertia and consists of the relative foundation free-field displacements and rotations described above.
3. Foundation deformations. Flexural, axial, and shear deformations of foundation elements occur as a result of loads applied by the superstructure and the supporting soil medium. Such deformations represent the seismic demand for which foundation components should be designed. These deformations can also significantly affect the overall system behavior, especially with respect to damping.

Chapter 19 treats only the inertial interaction effects (the first item above). Inertial interaction in buildings tends to be important for stiff structural systems (such as shear walls and braced frames), particularly where the foundation soil is relatively soft (i.e., Site Classes C to E). Kinematic interaction effects are neglected in these provisions. Foundation design is covered in Section 12.13.

The procedures in Chapter 19 are used to modify the fixed-base properties (period and damping) of a structural system. If fixed-base properties are obtained from an analytical model of the structure, the fixed-base properties correspond to a condition without soil springs. If soil springs are included in the analytical model of the structure, then the procedures given in Chapter 19 should not be used to modify the building period. The damping procedures in Chapter 19 could still be used in this case if the foundation springs are linear (thus introducing no damping) and there are no dashpots in parallel with the springs. In the remainder of this commentary, it is assumed that the structural period and damping ratio that are being modified for SSI effects correspond to a fixed-base condition.

In design procedures that use response spectra to establish design values of base shear (i.e., force-based methods such as those given in Chapter 12), the effects of inertial SSI on the seismic response of buildings is represented as a function of the ratio of flexible- to fixed-base first-mode natural period, $\tilde{T}_1/T_1$, and system damping, β_0, attributable to foundation-soil interaction. The flexible-base first-mode damping ratio, $\tilde{\beta}$, is calculated using Eq. 19.2-9. Figure C19.1-1 illustrates two possible effects of inertial SSI on the peak base shear, which is commonly computed from spectral acceleration at the first mode. The spectral acceleration for a flexible-based structure ($\tilde{S}_a = \tilde{C}_s/g$) is obtained by entering the spectrum drawn for effective damping ratio, $\tilde{\beta}$, at the corresponding elongated period, $\tilde{T}$. For buildings with periods greater than about 0.5 s, using $\tilde{S}_a$ in lieu of S_a (= C_s/g) typically reduces base shear demand, whereas in very stiff structures SSI can increase the base shear. Most equivalent lateral force methods feature a flat spectral shape at low periods that, when coupled with the requirement that $\tilde{\beta} > \beta$, results in modeling of inertial SSI that can only decrease the base shear demand.

The method given in Chapter 19 for evaluating inertial SSI effects is optional and has rarely been used in practice. There are several reasons for this. First, because the guidelines were written such that base shear demand can only decrease through consideration of SSI, SSI effects are ignored to be conservative. Second, many design engineers who have attempted to apply the method on projects have done so for major, high-rise buildings for which they felt that evaluating SSI effects could provide cost savings. Unfortunately, inertial interaction effects are negligible

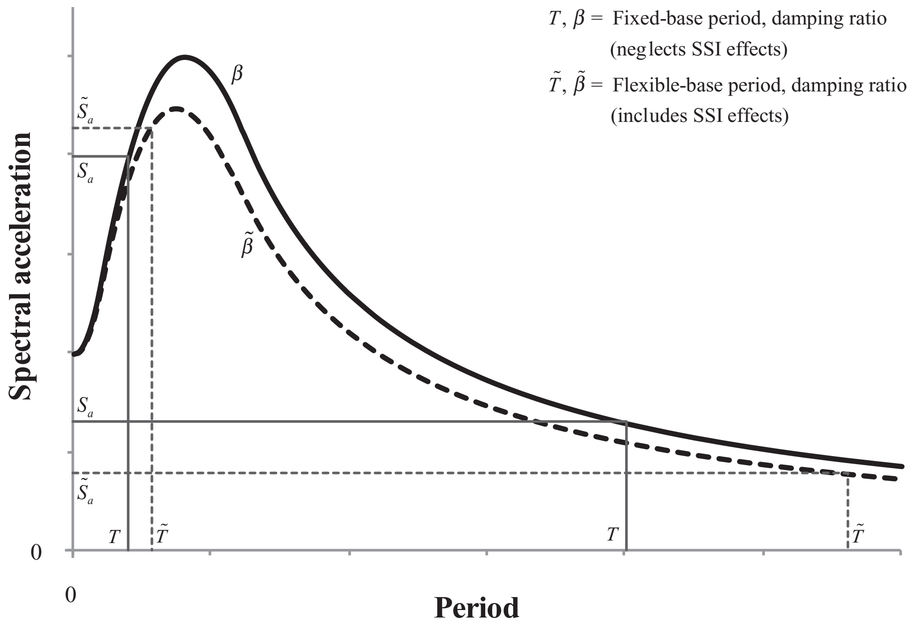

FIGURE C19.1-1 Schematic Showing Effects of Period Lengthening and Foundation Damping on Design Spectral Accelerations

for these tall, flexible structures, and hence the design engineers realized no benefit for their efforts and thereafter discontinued use of the procedures. The use of the procedures actually yield the most benefit for short-period (T < 1 s), stiff structures with stiff, interconnected foundation systems (i.e., mats or interconnected footings) founded on soil.

C19.2 EQUIVALENT LATERAL FORCE PROCEDURE

This procedure considers the response of the structure in its fundamental mode of vibration and accounts for the contributions of the higher modes to story shears implicitly through the choice of the effective weight of the structure and the vertical distribution of the lateral forces. The effects of soil-structure interaction are accounted for on the assumption that they influence only the contribution of the fundamental mode of vibration.

C19.2.1 Base Shear. Base shear is reduced for the effects of SSI as indicated in Eqs. 19.2-1 and 19.2-2. As indicated in Eq. 19.2-2, the change in base shear is related to the change in seismic coefficient (or spectral acceleration, as shown in Fig. C19.1-1). The term $(0.05/\tilde{\beta})^{0.4}$ in Eq. 19.2-2 represents the reduction in spectral ordinate associated with a change of damping from the fixed base value of $\beta = 0.05$ to the flexible base value of $\tilde{\beta}$.

C19.2.1.1 Effective Building Period. The fixed base period, T, is lengthened to the flexible-base period, $\tilde{T}$, using Eq. 19.2-3, which was derived by Veletsos and Meek (1974). Terms K_y and K_θ represent the translational and rocking stiffnesses of the foundation, respectively. The standard does not provide guidance on the evaluation of these stiffness terms. Equations for Ky and $K\theta$ are given by Gazetas (1991), and a number of practical considerations associated with the analysis of these terms are reviewed in FEMA 440 (2005). For convenience, simplified guidelines are presented in the following for these stiffness terms for use with the standard.

For building foundation systems that have lateral continuity, such as mats or footings interconnected with grade beams, stiffnesses K_y and K_θ can often be approximated as:

$$K_y = \frac{8}{2-\nu} G r_a \qquad \text{(C19.2-1)}$$

$$K_\theta = \frac{8}{3(1-\nu)} G r_m^3 \alpha_\theta \qquad \text{(C19.2-2)}$$

where

r_a = an equivalent foundation radius that matches the area of the foundation, A_0 (i.e., $r_a = \sqrt{(A_0/\pi)}$);

r_m = an equivalent foundation radius that matches the moment of inertia of the foundation in the direction of shaking (i.e., $r_m = \sqrt[4]{4I_0/\pi}$);

G = the strain-dependent shear modulus, as defined in the standard;

ν = the soil Poisson's ratio (generally taken as 0.3 for sands and 0.45 for clays); and

α_θ = a dimensionless coefficient that depends on the period of excitation, the dimensions of the foundation, and the properties of the supporting medium (Luco 1974, Veletsos and Verbic 1973, Veletsos and Wei 1971). A similar coefficient exists for translation (α_y) but can be taken as 1.0 without introducing significant error, and hence is not shown in Eq. C19.2-2.

As noted in the standard, shear modulus G is evaluated from small-strain shear wave velocity as $G = (G/G_o)G_o = (G/G_o)\gamma v_{so}^2/g$ (all terms defined in the standard). Shear wave velocity, v_{s0}, should be evaluated as the average small-strain shear wave velocity within the effective depth of influence below the foundation. The effective depth should be taken as $0.75r_a$ for horizontal vibrations of the foundation and $0.75r_m$ for rocking vibrations (Stewart et al. 2003). Methods for measuring v_{s0} (preferred) or estimating it from other soil properties are summarized elsewhere (e.g., Kramer 1996).

The dynamic modifier for rocking, α_θ, can significantly affect the computed response of some building structures. In the absence of more detailed analyses, for ordinary building structures with an embedment ratio $d/r_m < 0.5$ (where d = depth of embedment, measured from ground surface to base of foundation), the factor α_θ can be estimated as follows (Stewart et al. 2003):

$r_m/(v_{s0}T)$	α_θ
<0.05	1.0
0.15	0.85
0.35	0.7
0.5	0.6

Foundation embedment has the effect of increasing the stiffnesses K_y and K_θ. For embedded foundations for which there is positive contact between the side walls and the surrounding soil, K_y and K_θ may be determined from the following approximate formulas (Kausel 1974):

$$K_y = \frac{8Gr_a}{2-\nu}\left[1+\left(\frac{2}{3}\right)\left(\frac{d}{r_a}\right)\right] \tag{C19.2-4}$$

$$K_\theta = \frac{8Gr_m^3}{3(1-\nu)}\left[1+2\left(\frac{d}{r_m}\right)\right] \tag{C19.2-5}$$

Experimental studies and field performance data (Stokoe and Erden 1975, Stewart et al. 1999) indicate that the effects of foundation embedment are sensitive to the condition of the backfill and that judgment must be exercised in using Eqs. C19.2-4 and C19.2-5. For example, if contact is lost between the soil and basement walls, stiffnesses K_y and K_θ should be determined from the formulas for surface-supported foundations. More generally, the quantity d above should be interpreted as the effective depth of foundation embedment for the conditions that would prevail during the design earthquake ground motion.

The formulas for K_y and K_θ presented previously can be applied to most soil profiles in which soil shear wave velocity, v_{s0}, changes with depth. However, if the soil profile consists of a surface stratum of soil underlain by a much stiffer deposit with a shear wave velocity more than twice that of the surface layer, K_y and K_θ may be determined from the following two generalized formulas in which G is the shear modulus of the surface soil and D_s is the total depth of the stratum:

$$K_y = \frac{8Gr_a}{2-\nu}\left[1+\left(\frac{2}{3}\right)\left(\frac{d}{r_a}\right)\right]\left[1+\left(\frac{1}{2}\right)\left(\frac{r_a}{D_s}\right)\right]\left[1+\left(\frac{5}{4}\right)\left(\frac{d}{D_s}\right)\right] \tag{C19.2-6}$$

$$K_\theta = \frac{8Gr_m^3}{3(1-\nu)}\left[1+2\left(\frac{d}{r_m}\right)\right]\left[1+\left(\frac{1}{6}\right)\left(\frac{r_m}{D_s}\right)\right]\left[1+0.7\left(\frac{d}{D_s}\right)\right] \tag{C19.2-7}$$

These formulas are based on analyses of a stratum supported on a rigid base (Elsabee et al. 1977, Kausel and Roesset 1975) and apply for $r/D_s < 0.5$ and $d/r < 1$ (r taken as either r_a or r_m). The applicability of those rigid base solutions to practical situations (nonrigid geologic media) was evaluated by Stewart et al. (2003), resulting in the recommendations provided in the foregoing.

For buildings supported on footing foundations, these formulas can generally be used with r_a and r_m calculated using the full building footprint dimensions, provided that the footings are interconnected with grade beams. An exception can occur for buildings with both shear walls and frames, for which the rotation of the foundation beneath the wall may be independent of that for the foundation beneath the column (this type is referred to as weak rotational coupling). In such cases, r_m is often best calculated using the dimensions of the wall footing. Very stiff foundations, which provide strong rotational coupling, are best described using an r_m value that reflects the full foundation dimension. Regardless of the degree of rotational coupling, r_a should be calculated using the full foundation dimension if foundation elements are interconnected or continuous. Further discussion can be found in FEMA (2005). The use of discrete (noninterconnected) spread footing foundations in seismic regions is not recommended.

For buildings supported on pile foundations, lateral stiffness, K_y, can be taken as the sum of the lateral head stiffnesses of the supporting piles. These stiffness values are generally calculated using a beam on the Winkler foundation model, which is discussed in detail elsewhere (e.g., Salgado 2006). Rotational stiffness, K_θ, can be calculated from the vertical stiffness of the individual piles, k_{zi}, as follows:

$$K_\theta \approx \sum_i k_{zi} y_i^2 \tag{C19.2-8}$$

where y_i = horizontal distance from the foundation centroidal axis to pile i measured in the direction of shaking. The approximation in Eq. C19.2-8 assumes an infinitely rigid pile cap and neglects the rotational stiffness of individual piles, which is typically a small contribution. Quantity k_{zi} can be calculated for an individual pile using well established methods, such as discrete element modeling with *t-z* curves (Salgado 2006).

The alternate approach in the standard, represented by Eq. 19.2-5, was derived using Poisson's ratio $v = 0.4$, and is generally sufficient for nonembedded foundations that are laterally continuous across the building footprint and for which there is no "rigid" layer at depth in the profile (which would require the use of Eqs. C19.2-6 and C19.2-7 to calculate foundation stiffness). The value of the relative weight parameter, α (defined in the standard), can be taken as approximately 0.15 for typical buildings.

C19.2.1.2 Effective Damping. Bielak (1975 and 1976) and Veletsos and Nair (1975) expressed the flexible-base first-mode damping ratio, $\tilde{\beta}$, as indicated in Eq. 19.2-9. This equation is based on analyses of the harmonic response of single-degree-of-freedom oscillators supported on a viscoelastic medium with hysteretic damping. Foundation damping factor β_0 incorporates the effects of energy dissipation into the soil caused by radiation damping and hysteretic damping in the soil.

Figure 19.2-1 shows β_0 as a function of period lengthening ratio and was derived from the analytical solution presented in Veletesos and Nair (1975) for the condition of zero foundation embedment. Additional damping can be realized for embedded foundations, and the use of damping values from Fig. 19.2-1 is conservative for such conditions. More exact solutions can be obtained using procedures given in FEMA (2005).

Equation 19.2-9, in combination with the information presented in Fig. 19.2-1, may lead to damping factors for the soil-foundation-structure system, $\tilde{\beta}$, that are smaller than the fixed-base structural damping, β (assumed to be 0.05). However, it is recommended that $\tilde{\beta}$ never be taken as less than 0.05 for design applications. The use of values of $\tilde{\beta} > \beta$ is well justified from field case-history data (Stewart et al. 1999 and 2003).

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The presence of a stiff layer at depth in the soil profile can impede radiation damping, rendering the values in Fig. 19.2-1 too high. If a site consists of a relatively uniform layer of depth, *Ds*, overlying a very stiff layer with a shear wave velocity more than twice that of the surface layer, damping values should be reduced as indicated by Eq. 19.2-12.

C19.2.2 Vertical Distribution of Seismic Forces. The vertical distributions of the equivalent lateral forces for flexibly and rigidly supported structures are similar, and it is recommended that the same distribution be used in both cases, changing only the magnitude of the forces to correspond to the appropriate base shear. A greater degree of refinement in this step would be inconsistent with the approximations embodied in the requirements for rigidly supported structures.

With the vertical distribution of the lateral forces established, the overturning moments and the torsional effects about a vertical axis are computed as for rigidly supported structures. The above procedure is applicable to planar structures and, with some extension, to three-dimensional structures.

C19.2.3 Other Effects. In addition to its effect on base shear, inertial SSI also can increase the horizontal displacements of the structure relative to its base (because of rocking). This situation can increase the required spacing between structures and secondary design forces associated with P-delta effects. Such effects can be significant for stiff structural systems (e.g., walls and braced frames).

C19.3 MODAL ANALYSIS PROCEDURE

The procedure outlined in Section C19.2 is applicable to a modal analysis by adjusting the modal period and damping ratio of the fundamental mode only. Higher modes are relatively unaffected by SSI (Bielak 1976, Chopra and Gutierrez 1974, and Veletsos 1977). Hence, the contributions of higher modes are computed as if the structure were fixed at the base, and the maximum value of a response quantity is determined as for fixed-base structures but with the adjusted first-mode responses.

REFERENCES

American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI). (2007). "Seismic rehabilitation of existing buildings." *ASCE/SEI 41*, ASCE, Reston, Va.

Bielak, J. (1975). "Dynamic behavior of structures with embedded foundations." *Earthquake Engrg. and Struct. Dyn.*, 3, 259–274.

Bielak, J. (1976). "Modal analysis for building-soil interaction." *J. Engrg. Mech. Div.*, 102(5), 771–786.

Chopra, A. K., and Gutierrez, J. A. (1974). "Earthquake analysis of multistory buildings including foundation interaction." *J. Earthquake Engrg. and Struct. Dyn.*, 3, 65–67.

Elsabee, F., Kausel, E., and Roesset, J. M. (1977). "Dynamic stiffness of embedded foundations." *Advances in Civil Engineering through Engineering Mechanics*, ASCE, New York, 40–43.

Federal Emergency Management Agency (FEMA). (2005). "Improvement of nonlinear static seismic analysis procedures." *FEMA 440*. Federal Emergency Management Agency, Washington, D.C.

Gazetas, G. (1991). "Formulas and charts for impedances of surface and embedded foundations." *J. Geotech. Engrg.*, 117(9), 1363–1381.

Kausel, E. (1974). *Force Vibrations of Circular Foundations on Layered Media*, Report R74-11, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Mass.

Kausel, E., and Roesset, J. M. (1975). "Dynamic stiffness of circular foundations." *J. Engrg. Mech. Div.*, 101(6), 771–785.

Kramer, S. L. (1996). *Geotechnical Earthquake Engineering*, Prentice-Hall, Upper Saddle River, N.J.

Luco, J. E. (1974). "Impedance functions for a rigid foundation on a layered medium." *Nuclear Engrg. and Design*, (31), 204–217.

Salgado, R. (2006). *The Engineering of Foundations*. McGraw-Hill.

Stewart, J. P., Seed, R. B., and Fenves, G. L. (1999). "Seismic soil-structure interaction in buildings. II: Empirical findings." *J. Geotech. and Geoenvir. Engrg.*, 125(1), 38–48.

Stewart, J. P., Kim, S., Bielak, J., Dobry, R., and Power, M. (2003). "Revisions to soil structure interaction procedures in NEHRP design provisions." *Earthquake Spectra*, 19(3), 677–696.

Stokoe, K. H., II, and Erden, S. M. (1975). "Torsional response of embedded circular foundations." *Proceedings of the 5th European Conference on Earthquake Engineering*.

Veletsos, A. S. (1977). "Dynamics of structure-foundation systems." *Structural and Geotechnical Mechanics, A Volume Honoring N. M. Newmark*, W. J. Hall, ed., Prentice-Hall, Englewood Cliffs, N.J. 333–361.

Veletsos, A. S., and Meek, J. W. (1974). "Dynamic behavior of building-foundation systems." *Earthquake Engrg. and Struct. Dyn.*, 3(2), 121–138.

Veletsos, A. S., and Nair, V. V D. (1975). "Seismic interaction of structures on hysteretic foundations." *J. ASCE Struct. Div.*, 101(1), 109–129.

Veletsos, A. S., and Verbic, B. (1973). "Vibration of viscoelastic foundations." *Earthquake Engrg. and Struct. Dyn.*, 2(1), 87–105.

Veletsos, A. S., and Wei, Y. T. (1971). "Lateral and rocking vibration of footings." *J. Soil Mech. and Found. Div.*, 97(9), 1227–1248.

CHAPTER C20
SITE CLASSIFICATION PROCEDURE FOR SEISMIC DESIGN

C20.1 SITE CLASSIFICATION

Site classification procedures are given in Chapter 20 for the purpose of classifying the site and determining site coefficients and site-adjusted maximum considered earthquake ground motions in accordance with Section 11.4.3. Site classification procedures are also used to define the site conditions for which site-specific site response analyses are required to obtain site ground motions in accordance with Section 11.4.7 and Chapter 21.

C20.3 SITE CLASS DEFINITIONS

C20.3.1 Site Class F. Site conditions for which the site coefficients F_a and F_v in Tables 11.4-1 and 11.4-2 may not be applicable for site-response analyses required by Section 11.4.7. For short-period structures, it is permissible to determine values of F_a and F_v assuming that liquefaction does not occur because ground motion data obtained in liquefied soil areas during earthquakes indicate that short-period ground motions generally are attenuated because of liquefaction, whereas long-period ground motions may be amplified. This exception does not affect the requirements in Section 11.8 to assess liquefaction potential as a geologic hazard and to develop hazard mitigation measures as required.

Sections C20.3.2 through C20.3.5. These sections and Table 20.3-1 provide definitions for Site Classes A through E. Except for the additional definitions for Site Class E in Section 20.3.2, the site classes are defined fundamentally in terms of the average small-strain shear wave velocity in the top 100 ft (30 m) of the soil or rock profile. If shear wave velocities are available for the site, they should be used to classify the site. However, recognizing that in many cases shear wave velocities are not available for the site, alternative definitions of the site classes also are included. These definitions are based on geotechnical parameters: standard penetration resistance for cohesionless soils and rock, and standard penetration resistance and undrained shear strength for cohesive soils. The alternative definitions are intended to be conservative because the correlation between site coefficients and these geotechnical parameters is more uncertain than the correlation with shear wave velocity. That is, values of F_a and F_v tend to be smaller if the site class is based on shear wave velocity rather than on the geotechnical parameters. Also, the site class definitions should not be interpreted as implying any specific numerical correlation between shear wave velocity and standard penetration resistance or undrained shear strength.

Although the site class definitions in Sections 20.3.2 through 20.3.5 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 affect the site amplification.

The site class should reflect the soil conditions that 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 (for example, structures with shallow spread footings, with laterally flexible piles, or with basements where 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 may be 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 sites with sloping bedrock or highly variable soil deposits across the building area require careful study because 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 used 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 in a sedimentary basin, or 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.

C20.4 DEFINITION OF SITE CLASS PARAMETERS

Section 20.4 provides formulas for defining site classes in accordance with definitions in Section 20.3 and Table 20.3-1. Equation 20.4-1 is for determining the effective average small-strain shear wave velocity, $\bar{v}_s$, to a depth of 100 ft (30 m) at a site. This equation defines $\bar{v}_s$ as 100 ft (30 m) divided by the sum of the times for a shear wave to travel through each layer within the upper 100 ft (30 m), where travel time for each layer is calculated as the layer thickness divided by the small-strain 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 directly averaging the velocities of the individual layers.

For example, consider a soil profile that has four 25-ft-thick layers with shear wave velocities of 500, 1,000, 1,500, and 2,000 ft/s. The arithmetic average of the shear wave velocities

is 1250 ft/s (corresponding to Site Class C), but Eq. 20.4-1 produces a value of 960 ft/s (corresponding to Site Class D). The Eq. 20.4-1 value is appropriate because the four layers are being represented by one layer with the same wave passage time.

Equation 20.4-2 is for classifying the site using the average standard penetration resistance blow count, $\bar{N}$, for cohesionless soils, cohesive soils, and rock in the upper 100 ft (30 m). A method of averaging analogous to the method of Eq. 20.4-1 for shear wave velocity is used. The maximum value of N that may be used for any depth of measurement in soil or rock is 100 blows/ft. For the common situation where rock is encountered, the standard penetration resistance, N, for rock layers is taken as 100.

Equations 20.4-3 and 20.4-4 are for classifying the site using the standard penetration resistance of cohesionless soil layers, $\bar{N}_{ch}$, and the undrained shear strength of cohesive soil layers, $\bar{s}_u$, within the top 100 ft (30 m). These equations are provided as an alternative to using Eq. 20.4-2 for which N values in all geologic materials in the top 100 ft (30 m) are used. Where using Eqs. 20.4-3 and 20.4-4, only the respective thicknesses of cohesionless soils and cohesive soils within the top 100 ft (30 m) are used.

CHAPTER C21
SITE-SPECIFIC GROUND MOTION PROCEDURES FOR SEISMIC DESIGN

C21.0 GENERAL

Site-specific procedures for computing earthquake ground motions include dynamic site response analyses and probabilistic and deterministic seismic hazard analyses (PSHA and DSHA), which may include dynamic site response analysis as part of the calculation. Use of site-specific procedures may be required in lieu of the general procedure in Sections 11.4.1 through 11.4.6; Section C11.4.7 explains the conditions under which the use of these procedures is required. 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 analysis. For example, uncertainties may exist in seismic source location, extent and geometry; maximum earthquake magnitude; earthquake recurrence rate; ground motion attenuation; local site conditions, including soil layering and dynamic soil properties; and possible two- or three-dimensional wave-propagation effects. The use of peer review for a site-specific ground motion analysis is encouraged.

Site-specific ground motion analysis can consist of one of the following approaches: (a) PSHA and possibly DSHA if the site is near an active fault, (b) PSHA/DSHA followed by dynamic site response analysis, and (c) dynamic site response analysis only. The first approach is used to compute ground motions for bedrock or stiff soil conditions (not softer than Site Class D). In this approach, if the site consists of stiff soil overlying bedrock, for example, the analyst has the option of either (a) computing the bedrock motion from the PSHA/DSHA and then using the site coefficient (F_a and F_v) tables in Section 11.4.3 to adjust for the stiff soil overburden or (b) computing the response spectrum at the ground surface directly from the PSHA/DSHA. The latter requires the use of attenuation equations for computing stiff soil-site response spectra (instead of bedrock response spectra).

The second approach is used where softer soils overlie the bedrock or stiff soils. The third approach assumes that a site-specific PSHA/DSHA is not necessary but that a dynamic site response analysis should or must be performed. This analysis requires the definition of an outcrop ground motion, which can be based on the 5% damped response spectrum computed from the PSHA/DSHA or obtained from the general procedure in Section 11.4. A representative set of acceleration time histories is selected and scaled to be compatible with this outcrop spectrum. Dynamic site response analyses using these acceleration histories as input are used to compute motions at the ground surface. The response spectra of these surface motions are used to define a maximum considered earthquake (MCE) ground motion response spectrum.

The approaches described in the aforementioned have advantages and disadvantages. In many cases, user preference governs the selection, but geotechnical conditions at the site may dictate the use of one approach over the other. On the one hand, if bedrock is at a depth much greater than the extent of the site geotechnical investigations, the direct approach of computing the ground surface motion in the PSHA/DSHA may be more reasonable. On the other hand, if bedrock is shallow and a large impedance contrast exists between it and the overlying soil (i.e., density times shear-wave velocity of bedrock is much greater than that of the soil), the two-step approach might be more appropriate.

Use of peak ground acceleration as the anchor for a generalized site-dependent response spectrum is discouraged because sufficiently robust ground motion attenuation relations are available for computing response spectra in western U.S. and eastern U.S. tectonic environments.

C21.1 SITE RESPONSE ANALYSIS

C21.1.1 Base Ground Motions. Ground motion acceleration histories that are representative of horizontal rock motions at the site are required as input to the soil model. Where a site-specific ground motion hazard analysis is not performed, the MCE response spectrum for Site Class B (rock) is defined using the general procedure described in Section 11.4.1. If the model is terminated in material of Site Class A, C, or D, the input MCE response spectrum is adjusted in accordance with Section 11.4.3. The U.S. Geological Survey national seismic hazard mapping project website (http://earthquake.cr.usgs.gov/research/hazmaps/) includes hazard deaggregation options that can be used to evaluate the predominant types of earthquake sources, magnitudes, and distances contributing to the probabilistic ground motion hazard. Sources of recorded acceleration time histories include the databases of the Consortium of Organizations for Strong Motion Observation Systems (COSMOS) Virtual Data Center website (db.cosmos-eq.org), the Pacific Earthquake Engineering Research Center (PEER) Strong Motion Data Base website (http://peer.berkeley.edu/products/strong_ground_motion_db.html/), and the U.S. National Center for Engineering Strong Motion Data (NCESMD) website (http://www.strongmotioncenter.org). Ground motion acceleration histories at these sites generally were recorded at the ground surface and hence apply for an outcropping condition and should be specified as such in the input to the site response analysis code (Kwok et al. 2007 has additional details).

C21.1.2 Site Condition Modeling. Modeling criteria are established by site-specific geotechnical investigations that should include (a) borings with sampling; (b) standard penetration tests (SPTs), cone penetrometer tests (CPTs), and/or other subsurface investigative techniques; and (c) laboratory testing to

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establish the soil types, properties, and layering. The depth to rock or stiff soil material should be established from these investigations. Investigation should extend to bedrock or, for very deep soil profiles, to material in which the model is terminated. Although it is preferable to measure shear wave velocities in all soil layers, it is also possible to estimate shear wave velocities based on measurements 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).

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 may be significant (for example, sloping ground sites). 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 behavior 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 reductions of soil stiffness and strength. Typically, modulus reduction curves and damping curves are selected on the basis of published relationships for similar soils (for example, Vucetic and Dobry 1991, Electric Power Research Institute 1993, Darendeli 2001, Menq 2003, and Zhang et al. 2005). 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 based on field tests to determine these parameters or, if such tests are not possible, on published relationships and experience for similar soils in the local area. The uncertainty in the selected maximum shear moduli, modulus reduction and damping curves, and other soil properties should be estimated (Darendeli 2001 and Zhang et al. 2008). Consideration of the range of stiffnesses prescribed in Section 12.13.3 (increasing and decreasing by 50%) is recommended.

C21.1.3 Site Response Analysis and Computed Results. Analytical methods may be equivalently linear or nonlinear. Frequently used computer programs for one-dimensional analysis include the equivalent linear program SHAKE (Schnabel et al. 1972 and Idriss and Sun 1992) and the nonlinear programs FLAC (Itasca 2005); DESRA-2 (Lee and Finn 1978); MARDES (Chang et al. 1991); SUMDES (Li et al. 1992); DMOD_2 (Matasovic 2006); DEEPSOIL (Hashash and Park 2001); TESS (Pyke 2000); and OpenSees (Ragheb 1994, Parra 1996, and Yang 2000). If the soil response induces large strains in the soil (such as for 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) should be used (for example, FLAC, DESRA-2, SUMDES, D-MOD, TESS, DEEPSOIL, and OpenSees). Response spectra of output motions at the ground surface are calculated as the ratios of response spectra of ground surface motions to input outcropping rock motions. Typically, an average of the response spectral ratio curves is obtained and multiplied by the input MCE response spectrum to obtain the MCE ground surface response spectrum. Alternatively, the results of site-response analyses can be used as part of the PSHA using procedures described by Goulet et al. (2007) and programmed for use in OpenSHA (www.opensha.org; Field et al. 2005). Sensitivity analyses to evaluate effects of soil-property uncertainties should be conducted and considered in developing the final MCE response spectrum.

C21.2 RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION HAZARD ANALYSIS

Site-specific risk-targeted maximum considered earthquake (MCE_R) ground motions are based on separate calculations of site-specific probabilistic and site-specific deterministic ground motions.

Both the probabilistic and deterministic ground motions are defined in terms of 5% damped spectral response in the maximum direction of horizontal response. The maximum direction in the horizontal plane is considered the appropriate ground motion intensity parameter for seismic design using the equivalent lateral force (ELF) procedure of Section 12.8 with the primary intent of avoiding collapse of the structural system.

Most ground motion relations are defined in terms of average (geometric mean) horizontal response. Maximum response in the horizontal plane is greater than average response by an amount that varies with period. Maximum response may be reasonably estimated by factoring average response by period-dependent factors, such as 1.1 at short periods and 1.3 at a period of 1.0 s (Huang et al. 2008). The maximum direction was adopted as the ground motion intensity parameter for use in seismic design in lieu of explicit consideration of directional effects.

C21.2.1 Probabilistic (MCE_R) Ground Motions. Probabilistic seismic hazard analysis (PSHA) methods and subsequent computations of risk-targeted probabilistic ground motions based on the output of PSHA are sufficient to define MCE_R ground motion at all locations except those near highly active faults. Descriptions of current PSHA methods can be found in McGuire (2004). The primary output of PSHA methods is a so-called hazard curve, which provides mean annual frequencies of exceeding various user-specified ground motion amplitudes. Risk-targeted probabilistic ground motions are derived from hazard curves using one (or both for comparison purposes) of the methods described in the following two subsections.

C21.2.1.1 Method 1. The simpler but more approximate method of computing a risk-targeted probabilistic ground motion for each spectral period in a response spectrum is to first interpolate from a site-specific hazard curve the ground motion for a mean annual frequency corresponding to 2% probability of exceedance in 50 years (namely 1/2,475 per year). Then this "uniform-hazard" ground motion is factored by a so-called risk coefficient for the site location that is based on those mapped in Figs. 22-17 and 22-18. Via the method explained in the next subsection, the mapped risk coefficients have been computed from the USGS hazard curves for Site Class B and spectral periods of 0.2 and 1.0 s.

C21.2.1.2 Method 2. The direct method of computing risk-targeted probabilistic ground motions uses the entire site-specific hazard curve that results from PSHA. The computation is detailed in Luco et al. (2007). Summarizing, the hazard curve is

combined with a collapse fragility (or probability distribution of the ground motion amplitude that causes collapse) that depends on the risk-targeted probabilistic ground motion itself. The combination quantifies the risk of collapse. Iteratively, the risk-targeted probabilistic ground motion is modified until combination of the corresponding collapse fragility with the hazard curve results in a risk of collapse of 1% in 50 years. This target is based on the average collapse risk across the western United States that is expected to result from design for the probabilistic MCE ground motions in ASCE 7-10.

C21.2.2 Deterministic (MCE_R) Ground Motions. Deterministic ground motions are to be based on characteristic earthquakes on all known active faults in a region. 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.

For consistency, the same attenuation equations and ground motion variability used in the PSHA should be used in the deterministic seismic hazard analysis (DSHA). Adjustments for directivity/directional effects should also be made, when appropriate. In some cases, ground motion simulation methods may be appropriate for the estimation of long-period motions at sites in deep sedimentary basins or from great ($M \geq 8$) or giant ($M \geq 9$) earthquakes, for which recorded ground motion data are lacking.

C21.2.3 Site-Specific MCE_R. Because of the deterministic lower limit on the MCE_R spectrum (Figure 21.2-1), the site-specific MCE_R ground motion is equal to the corresponding risk-targeted probabilistic ground motion wherever it is less than the deterministic limit (e.g., 1.5g and 0.6g for 0.2 and 1.0 s, respectively, and Site Class B). Where the probabilistic ground motions are greater than the lower limits, the deterministic ground motions sometimes govern, but only if they are less than their probabilistic counterparts. On the MCE_R ground motion maps in ASCE/SEI 7-10, the deterministic ground motions govern mainly near major faults in California (like the San Andreas) and Nevada. The deterministic ground motions that govern are as small as 40% of their probabilistic counterparts.

C21.3 DESIGN RESPONSE SPECTRUM

Eighty percent of the design response spectrum determined in accordance with Section 11.4.5 was established as the lower limit to prevent the possibility of site-specific studies generating unreasonably low ground motions from potential misapplication of site-specific procedures or misinterpretation or mistakes in the quantification of the basic inputs to these procedures. Even if site-specific studies were correctly performed and resulted in ground motion response spectra less than the 80% lower limit, the uncertainty in the seismic potential and ground motion attenuation across the United States was recognized in setting this limit. Under these circumstances, the allowance of up to a 20% reduction in the design response spectrum based on site-specific studies was considered reasonable.

C21.4 DESIGN ACCELERATION PARAMETERS

The 90% lower limit rule, which can affect the determination of S_{DS}, was inserted because it was recognized that site-specific studies could produce response spectra with ordinates at periods greater than 0.2 s that were significantly greater than those at 0.2 s. Similarly, the rule that requires that S_{D1} be taken as the larger of the spectral acceleration at a period of 1 s and two times the spectral acceleration at a period of 2 s accounts for the possibility that the assumed $1/T$ proportionality for the constant velocity portion of the design response spectrum begins at periods greater than 1 s or is actually $1/T^n$ (where $n < 1$). Thus, this rule leads to more accurate spectral ordinates at periods around 2 s and conservative estimates at shorter periods. However, the conservatism is unlikely to be excessive.

C21.5 MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN (MCE_G) PEAK GROUND ACCELERATION

Site-specific requirements for determination of PGA are provided in a new Section 21.5 that is parallel to the procedures for developing site-specific response spectra in Section 21.2. The site-specific MCE peak ground acceleration, PGA_M, is taken as the lesser of the probabilistic geometric mean peak ground acceleration of Section 21.5.1 and the deterministic geometric mean peak ground acceleration of Section 21.5.2. Similar to the provisions for site-specific spectra, a deterministic lower limit is prescribed for PGA_M with the intent to limit application of deterministic ground motions to the site regions containing active faults where probabilistic ground motions are unreasonably high. However, the deterministic lower limit for PGA_M (in g) is set at a lower value, 0.5 F_{PGA}, than the value set for the zero-period response spectral acceleration, 0.6 F_a. The rationale for the value of the lower deterministic limit for spectra is based on the desire to limit minimum spectral values, for structural design purposes, to the values given by the 1997 Uniform Building Code (UBC) for Zone 4 (multiplied by a factor of 1.5 to adjust to the MCE level). This rationale is not applicable to PGA_M for geotechnical applications, and therefore a lower value of 0.5 F_{PGA} was selected. Section 21.5.3 of ASCE 7-10 states that the site-specific MCE peak ground acceleration cannot be less than 80% of PGA_M derived from the PGA maps. The 80% limit is a long-standing base for site-specific analyses in recognition of the uncertainties and limitations associated with the various components of a site-specific evaluation.

REFERENCES

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

Darendeli, M. (2001). "Development of a new family of normalized modulus reduction and material damping curves." Ph.D. Dissertation, Department of Civil Engineering, University of Texas, Austin.

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

Field, E. H., Gupta, N., Gupta, V., Blanpied, M., Maechling, P., and Jordan, T. H. (2005). "Hazard calculations for the WGCEP-2002 forecast using OpenSHA and distributed object technologies." *Seismological Res. Letters*, 76, 161–167.

Goulet, C. A., Stewart, J. P., Bazzurro, P., and Field, E. H. (2007). "Integration of site-specific ground response analysis results into probabilistic seismic hazard analyses." Paper 1486, *Proceedings of the 4th International Conference on Earthquake Geotechnical Engineering*, Thessaloniki, Greece.

Hashash, Y. M. A., and Park, D. (2001). "Non-linear one-dimensional seismic ground motion propagation in the Mississippi embayment." *Engrg. Geology*, 62(1–3), 185–206.

Huang, Y.-N., Whittaker, A. S., and Luco, N. (2008). "Maximum spectral demands in the near-fault region." *Earthquake Spectra*, 24(1), 319–341.

Minimum Design Loads for Buildings and Other Structures

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

Itasca Consulting Group. (2005). *FLAC*, *Fast Langrangian Analysis of Continua*, Version 5.0. Itasca Consulting Group, Minneapolis, Minn.

Kramer, S. L. (1996). *Geotechnical Earthquake Engineering,* Prentice-Hall.

Kwok, A. O., Stewart, J. P., Hashash, Y. M. A., Matasovic, N., Pyke, R., Wang, Z., and Yang, Z. (2007). "Use of exact solutions of wave propagation problems to guide implementation of nonlinear seismic ground response analysis procedures." *J. Geotech. and Geoenvir. Engrg.,* 133(11), 1385–1398.

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

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

Luco, N., Ellingwood, B. R., Hamburger, R. O., Hooper, J. D., Kimball, J. K., and Kircher, C. A. (2007). "Risk-targeted versus current seismic design maps for the conterminous United States." *Proceedings of the SEAOC 76th Annual Convention*. Structural Engineers Association of California, Sacramento, Calif.

Matasovic, N. (2006). "D-MOD_2–A computer program for seismic response analysis of horizontally layered soil deposits, earthfill dams, and solid waste landfills." *User's Manual*, GeoMotions, LLC, Lacey, Wash., 20 (plus Appendices).

McGuire, R. K. (2004). *Seismic Hazard and Risk Analysis*, Monograph, MNO-10. Earthquake Engineering Research Institute, Oakland, Calif., 221.

Menq, F. (2003). "Dynamic properties of sandy and gravely soils." Ph.D. Dissertation, Department of Civil Engineering, University of Texas, Austin.

Parra, E. (1996). "Numerical modeling of liquefaction and lateral ground deformation including cyclic mobility and dilation response in soil systems." Ph.D. Dissertation, Department of Civil Engineering, Rensselaer Polytechnic Institute, Troy, N.Y.

Pyke, R. M. (2000). "TESS: A computer program for nonlinear ground response analyses." TAGA Engineering Systems & Software, Lafayette, Calif.

Ragheb, A. M. (1994). "Numerical analysis of seismically induced deformations in saturated granular soil strata." Ph.D. Dissertation, Department of Civil Engineering, Rensselaer Polytechnic Institute, Troy, N.Y.

Schnabel, P. B., Lysmer, J., and Seed, H. B. (1972). *SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites*, Report EERC 72-12. Earthquake Engineering Research Center, University of California, Berkeley.

Vucetic, M., and Dobry, R. (1991). "Effect of soil plasticity on cyclic response." *J. Geotech. Engrg.*, 117(1), 89–107.

Yang, Z. (2000). "Numerical modeling of earthquake site response including dilation and liquefaction." Ph.D. Dissertation, Department of Civil Engineering and Engineering Mech., Columbia University, New York.

Zhang, J., Andrus, R. D., and Juang, C. H. (2005). "Normalized shear modulus and material damping ratio relationships." *J. Geotech. and Geoenvir. Engrg.*, 131(4), 453–464.

Zhang, J., Andrus, R. D., and Juang, C. H. (2008). "Model uncertainty in normalized shear modulus and damping relationships." *J. Geotech. and Geoenvir. Engrg.*, 134(1).

OTHER REFERNCES (NOT CITED)

Abrahamson, N. A. (2000). "Effects of rupture directivity on probabilistic seismic hazard analysis." *Proceedings of the Sixth International Conference on Seismic Zonation*, Palm Springs, Calif.

Somerville, P. G., Smith, N. F., Graves, R. W., and Abrahamson, N. A. (1997). "Modification of empirical strong ground motion attenuation relations to include the amplitude and duration effects of rupture directivity." *Seismological Res. Letters*, 68, 199–222.

CHAPTER C22
SEISMIC GROUND MOTION LONG-PERIOD TRANSITION AND RISK COEFFICIENT MAPS

RISK-ADJUSTED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTIONS MAPS

ASCE/SEI 7-10 continues to use contour maps of 0.2-s and 1-s spectral response accelerations to describe maximum considered earthquake (MCE) ground motions (Figs. 22-1 through 22-6). However, consistent with changes to the site-specific procedures of Section 21.2, the basis for the mapped values of the MCE_R ground motions in ASCE/SEI 7-10 is significantly different from that of the mapped values of MCE ground motions in previous editions of ASCE/SEI 7. These differences include use of (1) probabilistic ground motions that are risk-targeted (based on a uniform risk of collapse), rather than uniform hazard (uniform hazard exceedance probability); (2) deterministic ground motions that are based on the 84th percentile (approximately 1.8 times median), rather than 1.5 times median response spectral acceleration for sites near active faults; and (3) ground motion intensity that is based on maximum, rather than average (geometrical mean), response spectra acceleration in the horizontal plane. Except for determining the MCE_G peak ground acceleration (PGA), the mapped values are given as MCE_R spectral values.

The MCE_R ground maps incorporate new seismic hazard data developed by the U.S. Geological Survey (USGS) for the 2008 version of U.S. National Seismic Hazard Maps including new seismic, geologic, and geodetic information on earthquake rates and associated ground shaking (Petersen et al. 2008a and 2008b). These 2008 maps supersede versions released in 1996 and 2002.

The most significant changes to the 2008 maps fall into two categories, as follows:

1. Changes to earthquake source and occurrence rate models:
   - In California, the source model was updated to account for new information on faults. For example, models for the southern San Andreas Fault system were modified to incorporate new geologic data. The source model was also modified to better match the historical rate of magnitude 6.5 to 7 earthquakes.
   - The Cascadia Subduction Zone lying offshore of northern California, Oregon, and Washington was modeled using a distribution of large earthquakes between magnitude 8 and 9. Additional weight was given to the possibility for a catastrophic magnitude 9 earthquake that occurs, on average, every 500 years and results in fault rupture from northern California to Washington, compared with a model that allows for smaller ruptures.
   - The Wasatch Fault in Utah was modeled to include the possibility of rupture from magnitude 7.4 earthquakes on the fault.
   - Fault steepness estimates were modified based on global observations of normal faults.
   - Several new faults were included or revised in the Pacific Northwest, California, and the Intermountain West regions.
   - The New Madrid Seismic Zone in the central United States was revised to include updated fault geometry and earthquake information. In addition, the model was adjusted to include the possibility of several large earthquakes taking place within a few years or less, similar to the earthquake sequence of 1811–1812.
   - Source models for the region near Charleston, South Carolina, have been modified to include offshore faults that are thought to be capable of generating earthquakes.
   - A broader range of earthquake magnitudes was used for the central and eastern United States.
   - Earthquake catalogs and seismicity parameters were updated.
2. Changes to models of ground shaking (that show how ground motion decays with distance from an earthquake's source) for different parts of the United States, based on new published studies:
   - New NGA ground motion prediction models developed by the Pacific Earthquake Engineering Research Center (PEER) were adopted for crustal earthquakes beneath the western United States. These new models use shaking records from 173 global shallow crustal earthquakes to better constrain ground motion in western states.
   - Several new and updated ground-shaking models for earthquakes in the central and eastern United States were implemented in the maps. One of the new ground-shaking models accounts for the possibility that ground motion decays more rapidly from the earthquake source than was previously considered.
   - New ground motion models were applied for earthquake sources along the Cascadia Subduction Zone.

The new National Seismic Hazard Maps show, with some exceptions, similar or lower ground motion compared with the 2002 edition. For example, ground motion in the central and eastern United States has been generally lowered by about 10–25% because of the modifications of the ground motion models. Ground motion in the western United States is as much as 30% lower for shaking caused by long-period (1-s) seismic waves, and ground motion is similar (within 10–20%) for shaking caused by short-period (0.2-s) waves. Note, however, that the MCE_R ground motion maps derived from these USGS National Seismic Hazard Maps do not necessarily exhibit the same trends from ASCE/SEI 7-05 to ASCE/SEI 7-10 because of the

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aforementioned differences in the basis of the new MCE_R ground motion maps.

MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN (MCE_G) PGA MAPS

ASCE/SEI 7-10 now includes contour maps of maximum considered earthquake geometric mean (PGA_G) peak ground acceleration (Figs. 22-7 through 22-11) for use in geotechnical investigations (Section 11.8.3). In contrast to MCE_R ground motions, the maps of MCE_G PGA are defined in terms of geometric mean (rather than maximum direction) intensity and a 2% in 50-year hazard level (rather than a 1% in 50-year risk). Like the MCE_R ground motions, the maps of MCE_G PGA are governed near major active faults by deterministic values defined as 84th-percentile ground motions.

LONG-PERIOD TRANSITION MAPS

The maps of the long-period transition period, T_L, (Figs. 22-12 through 22-16) were introduced in ASCE/SEI 7-05. They were prepared by the USGS in response to Building Seismic Safety Council recommendations and subsequently included in the 2003 edition of the *Provisions*. See Section C11.4.5 for a discussion of the technical basis of these maps. The value of T_L obtained from these maps is used in Eq. 11.4-7 to determine values of S_a for periods greater than T_L.

The exception in Section 15.7.6.1, regarding the calculation of S_{ac}, the convective response spectral acceleration for tank response, is intended to provide the user the option of computing this acceleration with three different types of site-specific procedures: (a) the procedures in Chapter 21, provided they cover the natural period band containing T_c, the fundamental convective period of the tank-fluid system; (b) ground motion simulation methods using seismological models; and (c) analysis of representative accelerogram data. Elaboration of these procedures is provided below.

With regard to the first procedure, attenuation equations have been developed for the western United States (Next Generation Attenuation, e.g., Power et al. 2008) and for the central and eastern United States (e.g., Somerville et al. 2001) that cover the period band, 0 to 10 s. Thus, for $T_c \leq 10$ s, the fundamental convective period range for nearly all storage tanks, these attenuation equations can be used in the same probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) procedures described in Chapter 21 to compute $S_a(T_c)$. The 1.5 factor in Eq. 15.7-11, which converts a 5% damped spectral acceleration to a 0.5% damped value, could then be applied to obtain S_{ac}. Alternatively, this factor could be established by statistical analysis of 0.5% damped and 5% damped response spectra of accelerograms representative of the ground motion expected at the site.

In some regions of the United States, such as the Pacific Northwest and southern Alaska, where subduction-zone earthquakes dominate the ground motion hazard, attenuation equations for these events only extend to periods between 3 and 5 s, depending on the equation. Thus, for tanks with T_c greater than these periods, other site-specific methods are required.

The second site-specific method to obtain S_a at long periods is simulation through the use of seismological models of fault rupture and wave propagation (e.g., Graves and Pitarka 2004, Hartzell and Heaton 1983, Hartzell et al. 1999, Liu et al. 2006, and Zeng et al. 1994). These models could range from simple seismic source-theory and wave-propagation models, which currently form the basis for many of the attenuation equations used in the central and eastern United States, for example, to more complex numerical models that incorporate finite fault rupture for scenario earthquakes and seismic wave propagation through 2-D or 3-D models of the regional geology, which may include basins. These models are particularly attractive for computing long-period ground motions from great earthquakes ($M_w \geq \sim 8$) because ground motion data are limited for these events. Furthermore, the models are more accurate for predicting longer period ground motions because (a) seismographic recordings may be used to calibrate these models and (b) the general nature of the 2-D or 3-D regional geology is typically fairly well resolved at these periods and can be much simpler than would be required for accurate prediction of shorter period motions.

A third site-specific method is the analysis of the response spectra of representative accelerograms that have accurately recorded long-period motions to periods greater than T_c. As T_c increases, the number of qualified records decreases. However, as digital accelerographs continue to replace analog accelerographs, more recordings with accurate long-period motions are becoming available. Nevertheless, a number of analog and digital recordings of large and great earthquakes are available that have accurate long-period motions to 8 s and beyond. Subsets of these records, representative of the earthquake(s) controlling the ground motion hazard at a site, can be selected. The 0.5% damped response spectra of the records can be scaled using seismic source theory to adjust them to the magnitude and distance of the controlling earthquake. The levels of the scaled response spectra at periods around T_c can be used to determine S_{ac}. If the subset of representative records is limited, then this method should be used in conjunction with the aforementioned simulation methods.

RISK COEFFICIENT MAPS

The risk coefficient maps in ASCE/SEI 7-10 (Figs. 22-17 through 22-18) provide factors, C_{RS} and C_{R1}, that are used in the site-specific procedures of Chapter 21 (Section 21.2.1.1 Method 1). These factors are implicit in the MCE_R ground motion maps.

The mapped risk coefficients are the ratios of risk-targeted probabilistic ground motions (for 1% in 50 years collapse risk) derived from the 2008 USGS National Seismic Hazard Maps to corresponding uniform-hazard (2% in 50 years ground motion exceedance probability) ground motions. The computation of risk-targeted probabilistic ground motions is very briefly explained in Method 2 (Section 21.2.1.2) of the site-specific procedures of Chapter 21 and its commentary. Luco et al. (2007) has more information on the development of risk-targeted probabilistic ground motions and resultant risk coefficients.

GROUND MOTION SOFTWARE TOOL

The USGS has developed a companion software program that calculates location-specific spectral values based on latitude and longitude, address, or zip code; use of zip codes is discouraged in regions where ground motion values vary substantially over a short distance. The calculated values are based on the data used to prepare the maps. The spectral values can be adjusted for site class effects within the program using the site classification procedure in Chapter 20 and the site coefficients in Section 11.4. The companion software program may be accessed at the USGS website (http://earthquake.usgs.gov/designmaps) or through the SEI website at http://content.seinstitute.org. The software program should be used to establish spectral values for design because the maps found in ASCE/SEI 7-10 are too small to provide accurate spectral values for many sites.

REFERENCES

Graves, R. W., and Pitarka, A. (2004). "Broadband time history simulation using a hybrid approach." Paper 1098, *Proceedings of the 13th World Conference on Earthquake Engineering*, Vancouver, Canada.

Hartzell, S., and Heaton, T. (1983). "Inversion of strong ground motion and teleseismic waveform data for the fault rupture history of the 1979 Imperial Valley, California, earthquake." *Bull. Seismological Soc. of Am.*, 73, 1553–1583.

Hartzell, S., Harmsen, S., Frankel, A., and Larsen, S. (1999). "Calculation of broadband time histories of ground motion: Comparison of methods and validation using strong ground motion from the 1994 Northridge earthquake." *Bull. Seismological Soc. of Am.*, 89, 1484–1504.

Liu, P., Archuleta, R. J., and Hartzell, S. H. (2006). "Prediction of broadband ground-motion time histories: Hybrid low/high-frequency method with correlated random source parameters." *Bull. of the Seismological Soc. of Am.*, 96, 2118–2130.

Luco, N., Ellingwood, B. R., Hamburger, R. O., Hooper, J. D., Kimball, J. K., and Kircher, C. A. (2007). "Risk-targeted versus current seismic design maps for the conterminous United States." *Proceedings of the SEAOC 76th Annual Convention*. Structural Engineers Association of California, Sacramento, Calif.

Petersen, M. D., Frankel, A. D., Harmsen, S. C., Mueller, C. S., Haller, K. M., Wheeler, R. L., Wesson, R. L., Zeng, Y., Boyd, O. S., Perkins, D. M., Luco, N., Field, E. H., Wills, C. J., and Rukstales, K. S. (2008a). "Documentation for the 2008 update of the United States National Seismic Hazard Maps." USGS Open File Report 2008-1128.

Petersen, M. D., and others. (2008b). "2008 United States National Seismic Hazard Maps," U.S. Geological Survey Fact Sheet 2008-3018, 2.

Power, M., Chiou, B., Abrahamson, N., Bozorgnia, Y., Shantz, T., and Roblee, C. (2008). "An overview of the NGA project." *Earthquake Spectra Special Issue on the Next Generation of Ground Motion Attenuation (NGA) Project*. Earthquake Engineering Research Institute.

Somerville, P. G., Collins, N. Abrahamson, N., Graves, R., and Saikia, C. (2001). *Earthquake Source Scaling and Ground Motion Attenuation Relations for the Central and Eastern United States*, Final Report to the USGS under Contract 99HQGR0098.

Zeng, Y., Anderson, J. G., and Yu, G. (1994). "A composite source model for computing synthetic strong ground motions." *Geophys. Research Letters*, 21, 725–728.

Minimum Design Loads for Buildings and Other Structures