Significant changes from ASCE 7-05 to ASCE 7-10, part 1: Seismic design provisions

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- This paper presents the major changes that have taken place in the seismic design provisions from ASCE (American Society of Civil Engineers) 7-05 to ASCE 7-10, which is referenced by the 2012 *International Building Code*. Changes to the seismic hazard maps are presented, along with explanations as to why they were necessary and how they will affect seismic design.
- Other significant changes include major changes in the design force requirements for anchorage between walls and diaphragms providing lateral support, changes in the rules governing combinations of structural systems, increased height limits for structural systems, and changes in the approximate fundamental period for eccentrically braced frame and buckling-restrained braced frame systems.

The ASCE 7 standard *Minimum Design Loads for Buildings and Other Structures* is the document that the *International Building Code* (IBC) relies on for its structural provisions. ASCE 7-05,[1] the standard referenced in the 2006[2] and 2009[3] editions of the IBC, did not undergo the recently usual three-year update. In the last-published edition, ASCE (American Society of Civil Engineers) 7-10,[4] referenced by the 2012 IBC,[5] major revisions have taken place in wind design, seismic design, and other provisions from ASCE 7-05. The changes in the seismic design provisions are the focus of part 1 of this paper. Changes in wind-related provisions will be published in part 2, and the other changes in part 3. Changes in chapter 13, "Seismic Design Requirements for Nonstructural Components;" chapter 14, "Material Specific Seismic Design and Detailing Requirements;" and chapter 15, "Seismic Design Requirements for Nonbuilding Structures," are excluded from the scope of this paper.

There is little in ASCE 7 that relates exclusively to precast concrete. However, the changes are of interest to all, including designers of precast concrete.

Ground motion maps

Four significant changes have been made to the seismic ground motion maps:[6]

- The U.S. Geological Survey has made some changes in source zone modeling and in the attenuation relationships used. Source zone refers to tectonic faults and other geologic features, such as subduction zones, which can generate earthquakes. An attenuation relationship, also called ground motion prediction equation, describes how ground motion decays as it travels from source to site.

- Uniform-hazard ground motion has now been replaced by risk-targeted ground motion by switching from a 2% in 50-year hazard level to a 1% in 50-year collapse risk target. The risk-targeted maximum considered earthquake (MCE) ground motion is designated MCE_R ground motion.

- Geomean ground motions have been replaced by maximum-direction ground motions.

- Deterministic ground motions have been changed from 150% of median ground motions to 84th percentile ground motions, which are 180% of median ground motions. Note that geomean, rather than risk-targeted MCE ground motion, is required to be used for analysis of liquefaction potential by ASCE 7-10. Geomean MCE ground motion is designated MCE_G ground motion.

Electronic values of mapped acceleration parameters and other seismic design parameters are provided at https://geohazards.usgs.gov/secure/designmaps/us/ for the United States and https://geohazards.usgs.gov/secure/designmaps/ww/ for all other locations.

U.S. Geological Survey updates

The U.S. Geological Survey has updated some source zone models and has used next-generation attenuation relationships[7] instead of the old attenuation relationships in the western United States and new attenuation relationships in addition to the old relationships in the central and eastern United States.[6] The new relationships apparently show that eastern earthquakes are much more like western earthquakes than previously thought, with ground motion intensity dropping off more steeply with distance from the source than indicated by earlier attenuation curves. As a result, ground motion (particularly long-period ground motion) has decreased 50% or more in many parts of the United States.

Risk-targeted ground motion

The probabilistic portions of the MCE ground motion maps in the 1997,[8] 2000,[9] and 2003 National Earthquake Hazards Reduction Program *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*[10] and all editions of the IBC provide ground motion values that have a 2% probability of being exceeded in 50 years. It ought to be remembered that mapped values of ground motions are governed by probabilistic response spectral accelerations except at high-hazard sites located relatively close to active faults, where deterministic values govern.

As explained in the Commentary to the 2003 NEHRP provisions, while this approach provides for a uniform likelihood throughout the nation that the ground motion will not be exceeded, it does not provide for a uniform probability of failure for structures designed for that ground motion. There are significant variations in the probability of collapse because of uncertainty in the collapse capacity or factor of safety against collapse relative to the ground motion for which the structure is designed. These variations are particularly significant between locations in the western versus central and eastern United States.

Luco et al.[11] pointed out that use of the ASCE 7-05 seismic design maps would result in structures with uniform collapse probability within the probabilistic portions of the maps if the collapse capacity were not uncertain. They discuss sources of uncertainty in collapse capacity and quantitative estimation of its magnitude. Adjustments to the ground motion values on the ASCE 7 design maps that result in structures with uniform collapse probabilities (1% in 50-year collapse risk target) are demonstrated.

Relative to the probabilistic MCE ground motions in ASCE 7-05, the risk-targeted ground motions for design are as much as 30% smaller in the New Madrid, Mo., seismic zone; near Charleston, S.C.; and in the coastal region of Oregon, with less than 15% change almost everywhere else in the 48 contiguous states.

Maximum-direction ground motion

The procedure used to develop the statistical estimate of ground motion in the past resulted in the geometric mean (geomean) of two orthogonal components of motion at a site.

In the Applied Technology Council (ATC)-63 study of low-rise wood buildings by Filiatrault,[12] the overall failure rate for three-dimensional (3-D) analyses was higher than those for two-dimensional (2-D) analyses for the same set of structures analyzed for the same 22 pairs of ground motions. The specification of maximum-direction ground motions reduces the probability of structural failure based on equivalent static 2-D design, compared with the use of the geomean-based demand. For consistency, revisions have been made to both probabilistic and deterministic ground motion criteria to reflect required use of maximum-direction ground motions.

Huang et al.[13] found that near-source ground motion spectral response accelerations of the next-generation

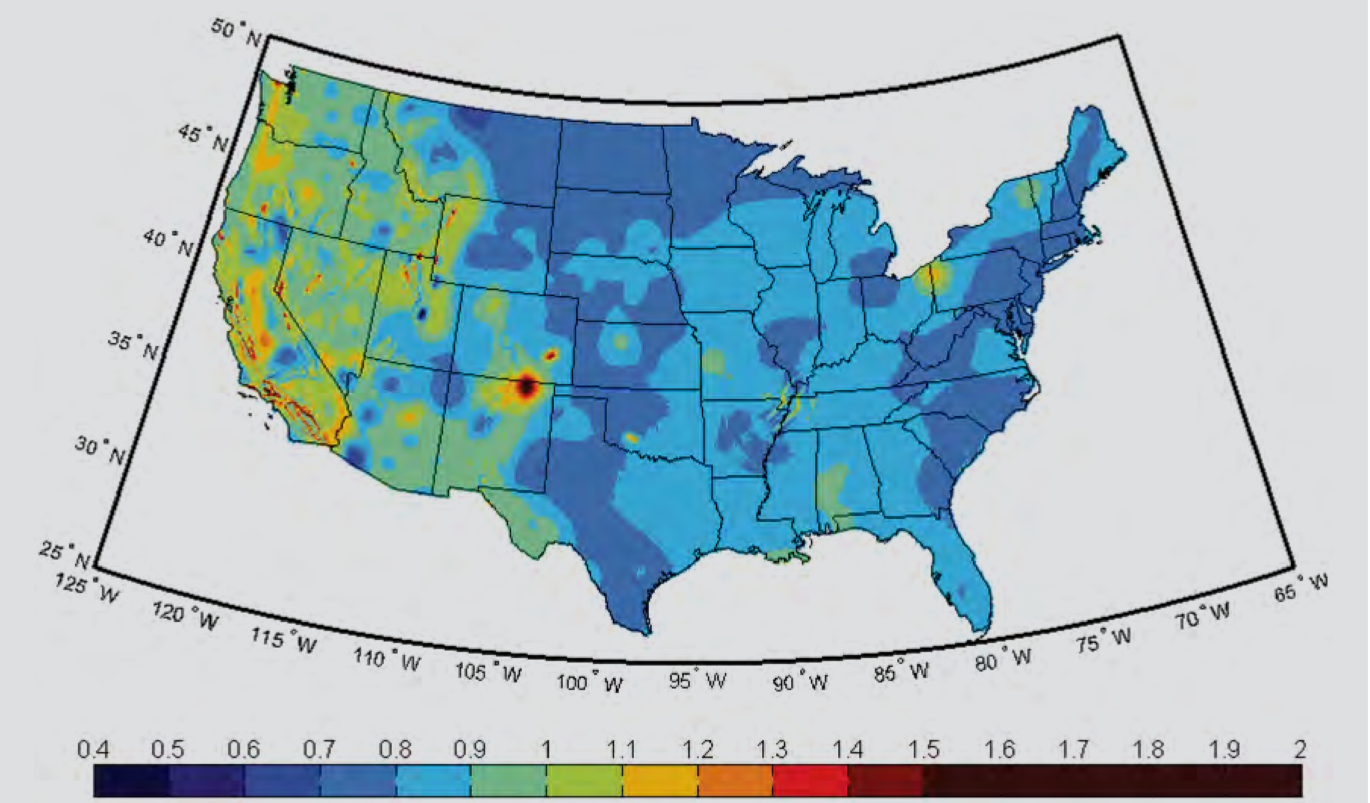

Figure 1. Ratio of ASCE 7-10 S_S-values to ASCE 7-05 S_S-values. Note: S_S = mapped MCE_R, 5%-damped, spectral response acceleration parameter at short periods.

attenuation relations are somewhat less than those in the maximum direction of response. Huang et al.[13] focused on earthquakes with moment magnitudes greater than 6.5 and site-to-source distances less than 15 km (9.3 mi). For this family of earthquake records, ground motions in the maximum direction of response were about 110% of 5%-damped, short-period spectral response acceleration, and about 130% of 5%-damped, 1-second spectral response acceleration calculated using the next-generation alternation relations.

Other regions (such as the central and eastern United States) are expected to have similar ratios of maximum-direction ground motions to geomean ground motions, though the limited number of strong-motion records from the central and eastern United States precludes rigorous evaluation. Beyer and Bommer,[14] using a set of 949 earthquake records with much wider ranges of moment magnitude (4.2 to 7.9) and hypocentral distance (5 to 200 km [3 to 120 mi]), found similar ratios of maximum to geomean response to those of Huang et al.[13] on large-magnitude, near-fault ground motions. The Beyer and Bommer data set[14] included records from more than 20 European earthquakes.

Maximum-direction values were thus obtained from geomean values of mapped MCE_R, 5%-damped, spectral response acceleration parameter at short periods S_S and mapped MCE_R, 5%-damped, spectral response acceleration parameter at a period of 1 second S_1 by applying scalar multiplication by factors of 1.1 and 1.3, respectively.

Deterministic ground motions

Deterministic ground motions should account for uncertainties associated with near-fault ground motions, particularly at longer periods, and necessitate a more statistically appropriate estimate of 5%-damped spectral response accelerations than those based on the 150% of the median ground motions used in ASCE 7-05. The use of 84th-percentile ground motions in ASCE 7-10 effectively requires increasing median ground motions by 180%. The technical basis of this change can be found in Huang et al.[13] They found 150% of the median spectral response accelerations of the new next-generation attenuation relations (average of three relations) to be significantly less than 84th-percentile ground motions in the maximum direction of response. Near active sources (in the western United States), 84th percentile ground motion in the maximum direction of response is about 200% (1.8 × 110%) of 5%-damped, short-period geomean spectral response acceleration and about 230% (1.8 × 130%) of 5%-damped, 1-second geomean spectral response acceleration of the new next-generation attenuation relations (average value of the three relations).

Observations

The following observations can be made by comparing the design ground motions of ASCE 7-10 with those of ASCE 7-05 (**Fig. 1** and **2** show a comparison between ASCE 7-10 and ASCE 7-05 mapped S_s and S_1 values):

- On a regional basis, the changes from ASCE 7-05 to ASCE 7-10 result in only a slight increase or decrease in design ground motions, on average. Notable exceptions are short-period ground motions in the central and eastern United States, changes in which substantially reduce design values. Other exceptions are for certain cities, such as St. Louis, Mo.; Chicago, Ill.; and New York, N.Y., where the changes would also reduce the seismic design category.

- In the western United States, the changes from ASCE 7-05 to ASCE 7-10 result in an increase or decrease of 10% or less in design ground motions.

- For certain cities, the changes from ASCE 7-05 to ASCE 7-10 substantially change design ground motions, primarily due to changes in underlying hazard functions. For instance, there have been sizable increases in San Bernardino, Calif., (S_S +39%, S_1 +57%) and significant decreases in the San Diego, Calif., area (S_S -22%, S_1 -23%).

MCE_G peak ground acceleration, liquefaction potential evaluation

ASCE 7-10 section 11.8.3, "Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F," has been modified to require evaluation of liquefaction potential for maximum considered earthquake ground motions, rather than design earthquake ground motions, to ensure that the potential occurrence and effects of liquefaction during the MCE are considered in geotechnical and structural design.

The provision also requires liquefaction potential evaluation using mapped peak ground acceleration (maps provided in ASCE 7-10 Fig. 22-7 through 22-11) adjusted for site effects, rather than using the ASCE 7-05 approximation for peak ground acceleration equal to the short-period spectral acceleration multiplied by a factor of 0.4. The new maps provide substantially more accurate values for peak ground acceleration because they are based on peak ground acceleration attenuation relationships. Peak ground acceleration is modified for site class effects using ASCE 7-10 Eq. (11.8-1) where the site coefficient F_{PGA} is obtained from ASCE 7-10 Table 11.8-1. Values of F_{PGA} in the table are identical to the short-period site coefficient F_a in ASCE 7-10 Table 11.4-1 but are a function of peak ground acceleration rather than S_S.

The mapped peak ground accelerations in ASCE 7-10 Fig. 22-7 through 22-11 are geomean values and not risk-targeted values. Thus, these are designated as MCE_G peak ground accelerations, unlike the spectral accelerations in ASCE 7-10 Fig. 22-1 through 22-6, which represent risk-targeted MCE or MCE_R ground motion.

There are three newly added sections in ASCE 7-10: 21.5.1 "Probabilistic MCE_G Peak Ground Acceleration," 21.5.2

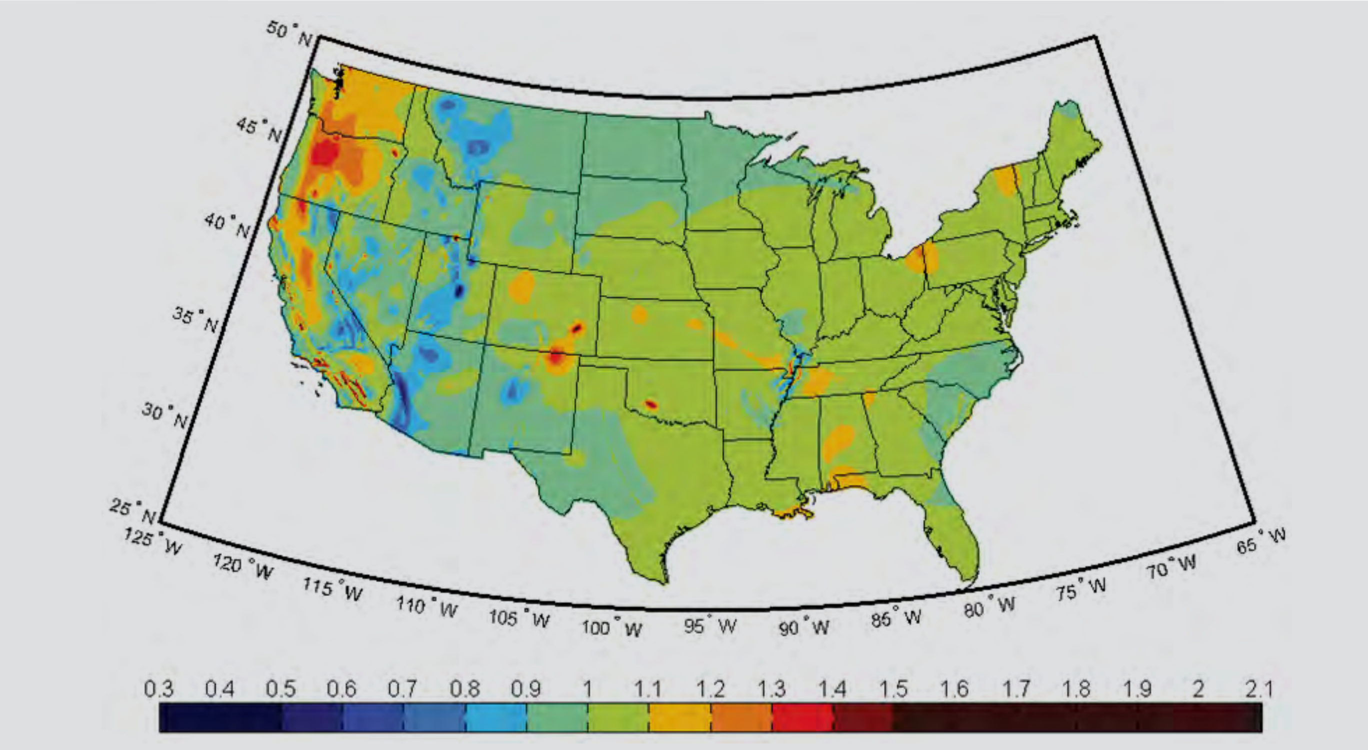

Figure 2. Ratio of ASCE 7-10 S_1-values to ASCE 7-05 S_1-values. Note: S_1 = mapped MCE_R, 5%-damped, spectral response acceleration parameter at a period of 1 second.

"Deterministic MCE_G Peak Ground Acceleration," and 21.5.3 "Site-Specific MCE_G Peak Ground Acceleration." These parallel ASCE 7-10 sections 21.2.1 "Probabilistic [MCE_R] Ground Motions," 21.2.2 "Deterministic [MCE_R] Ground Motions," and 21.2.3 "Site-Specific MCE_R," respectively. In ASCE 7-10 section 21.5.2, $0.5F_{PGA}$ is the deterministic lower limit on peak ground acceleration, where $0.5g$ is the bedrock peak ground acceleration and F_{PGA} is the site coefficient. A bedrock peak ground acceleration of $0.6g$ would have been the exact equivalent of the lower-bound limits of $1.5g$ and $0.6g$ on S_S and S_1, respectively, in ASCE 7-10 section 21.2.2. There was some objection to the $0.6g$ lower limit as putting a constraint on liquefaction analysis where there had previously been no limit. Some felt that the lower bounds on S_S and S_1 had their origin in structural behavior and should not apply to liquefaction. The $0.5g$ (rather than $0.6g$) was considered more appropriate as the lower limit on bedrock acceleration based on discussions within the Seismic Subcommittee of ASCE 7.

Changes in seismic design requirements for building structures

Changes in ASCE 7-10 chapter 11 that are not strictly related to earthquake ground motion and all chapter 12 changes are discussed in this section.

Structural integrity

ASCE 7-10 section 11.7 "Design Requirements for Seismic Design Category A" is now greatly reduced in size. Much of the contents of ASCE 7-05 section 11.7 have been transferred in modified form to ASCE 7-10 section 1.4 "General Structural Integrity" (**Fig. 3**). The latter was considered to be a more logical location. **Table 1** shows where the provisions in ASCE 7-05 section 11.7 are relocated in ASCE 7-10.

Classification of a building as nonbuilding structure

The following wording has been added to ASCE 7-10 section 11.1.3 "Applicability": "Buildings whose purpose is to enclose equipment or machinery and whose occupants are engaged in maintenance or monitoring of that equipment, machinery or their associated processes shall be permitted to be classified as nonbuilding structures designed and detailed in accordance with Section 15.5 of this standard." Wording has been added to section 11.1.3 of the ASCE 7-10 Commentary, which states that examples of such structures include, but are not limited to, boiler buildings, aircraft hangars, steel mills, aluminum smelting facilities, and other automated manufacturing facilities.

Table 1: Relocation of ASCE 7-05 section 11.7 to ASCE 7-10 section 1.4

ASCE 7-05	ASCE 7-10
11.7.2 Lateral Forces	1.4.3 Lateral Forces (modified)
11.7.3 Load Path Connections	1.4.2 Load Path Connections (modified)
11.7.4 Connection to Supports.	1.4.4 Connection to Supports (modified)
11.7.5 Anchorage of Concrete or Masonry Walls	1.4.5 Anchorage of Structural Walls (modified)

Design coefficients and factors for seismic-force-resisting systems

The following significant changes have been made in ASCE 7-10 Table 12.2-1:

- The material of construction now is almost always at the beginning of the description of a seismic-force-resisting system. For instance, it is now "steel special concentrically braced frames" rather than "special steel concentrically braced frames."
- Under "Bearing Wall Systems" and "Building Frame Systems," "Light-frame walls sheathed with wood structural panels rated for shear resistance or steel sheets" are now divided into two items: wood and cold-formed steel. The design coefficients and factors are not different for the two systems, but the referenced chapter 14 section numbers (column 2) are different. Also, "Light-frame wall systems using flat strap bracing" are now specifically indicated

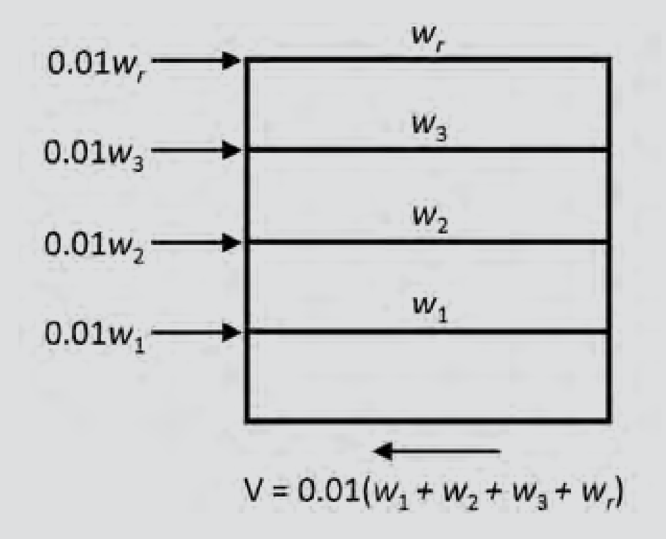

Figure 3. General structural integrity requirement of ASCE 7-10 section 1.4.3. Note $F_x = 0.01w_x$ = portion of the seismic base shear V induced at level x; V = design base shear; W = effective seismic weight of the building as defined in ASCE 7-10 section 12.7.2; w_r = portion of W that is located at or assigned to roof level; w_x = portion of W that is located at or assigned to level x.

Figure 4. Example of a cold-formed steel–special bolted moment frame.

to be of cold-formed steel under "Bearing Wall Systems."

- ASCE 7-05 included two different types of systems for eccentrically braced frames as well as buckling restrained braced frames under "Building Frame Systems." The primary distinction between the two types was whether there were moment-resisting beam-column connections within the braced bays. ASCE 7-10 consolidates the eccentrically braced frame and buckling restrained braced frame systems into a single designation with proper consideration of the beam-column connection demands. The change allows the engineer to either provide a fully restrained moment connection meeting the requirements for ordinary moment connections in American Institute of Steel Construction (AISC) 341 *Seismic Provisions for Structural Steel Buildings*[15] (thereby directly providing a load path to resist the connection force and deformation demands) or to provide a connection with the ability to accommodate the potential rotation demands.

- Seismic design factors and height restrictions for bearing wall systems consisting of ordinary reinforced and ordinary plain autoclaved aerated concrete masonry shear walls have been added to ASCE 7-10 Table 12.2-1. The values and restrictions are consistent with those in 2009 IBC section 1613.6.4.

- A newly defined seismic-force-resisting system titled "Cold-Formed Steel Special Bolted Moment Frame" (CFS-SBMF) has been introduced in Table 12.2-1 (**Fig. 4**). Also, the first edition of American Iron and Steel Institute (AISI) S110 *Standard for Seismic Design of Cold-Formed Steel Structural Systems—Special Bolted Moment Frames*,[16] which is based on research, has been adopted. The standard includes design provisions for the new system, which is expected to undergo large inelastic deformations during major seismic events. It is intended that most of the inelastic deformations will take place at the bolted connections due to slip and bearing. To develop the designated mechanism, requirements based on capacity design principles are provided for the design of the beams, the columns, and the associated connections. The response modification coefficient *R* is set at 3.5. The height limitation of 35 ft (11 m) for all seismic design categories (SDCs) is based on practical use only and not on any limits on the CFS-SBMF system strength.

Vertical combination of structural systems

When different lateral-force-resisting systems are vertically stacked, the ASCE 7-05 rule concerning seismic design coefficients was that the *R*-value could not increase and the

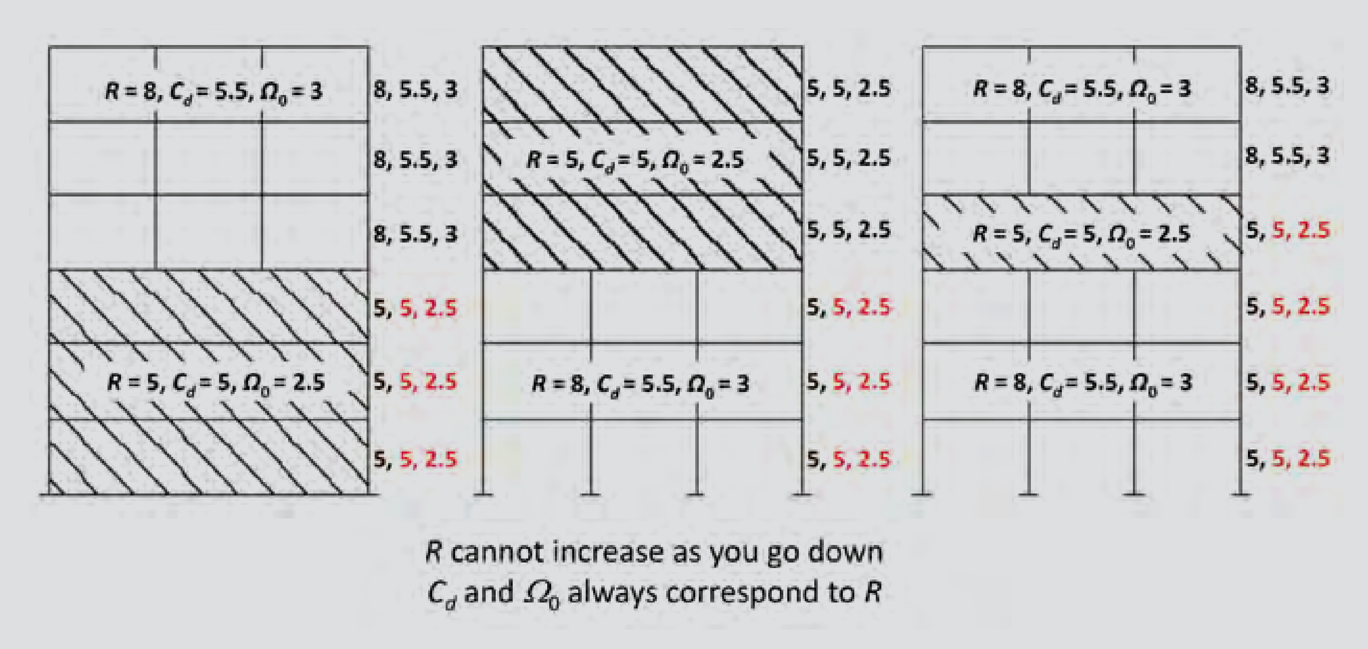

Figure 5. Revised vertical combination requirement. Note: C_d = deflection amplification factor; R = response modification coefficient; Ω_0 = overstrength factor.

values of overstrength factor Ω_0 and deflection amplification factor C_d could not decrease from the upper stories to the lower stories in a building. According to ASCE 7-10, the R-value still cannot increase from the upper stories to the lower stories. However, Ω_0 and C_d now must correspond to the R-value (**Fig. 5**).

Two-stage analysis procedure

The location of the base in condition (b) of the two-stage equivalent lateral force procedure is clarified. ASCE 7-10 section 11.2 defines *base* as the "level at which the horizontal seismic ground motions are considered to be imparted to the structure." Condition (b) of the two-stage equivalent lateral force procedure intends to reference the base of the upper portion of the structure, not the base of the entire structure. The definition of *base*, however, applies to the entire structure.

Item e in ASCE 7-10 section 12.2.3.2 was added to clarify that a static or dynamic analysis can be performed on the upper portion and that a static analysis is to be performed on the lower portion (**Fig. 6**). Because the lower portion is stiff, its seismic response will be dominated by the fundamental mode, which makes equivalent static analysis the logical choice.

Steel cantilever column systems

ASCE 7-05 contained provisions for steel ordinary, intermediate, and special cantilever column systems. In previous editions, AISC 341[17] did not explicitly address cantilever column systems. Consequently, the resulting set of requirements associated with each system was vague, confusing, and potentially incomplete. Modifications made to ASCE 7-10 section 12.2.5.2 and Table 12.2-1 have been coordinated with parallel changes in the 2010 edition of AISC 341.[15] AISC 341-10 does not have separate requirements for intermediate cantilevered column systems. Consequently, this system has been removed.

The reduction in the overstrength factor Ω_0 permitted by footnote g of ASCE 7-10 Table 12.2-1 is clarified. Neither the reduction by subtracting $^1/_2$ nor the 2.0 limit applies to cantilevered column systems, for which the value of Ω_0 is $1^1/_4$ or $1^1/_2$. Also, the word *one-half* was confusing and could be erroneously construed to mean one half of Ω_0 rather than the value of $^1/_2$.

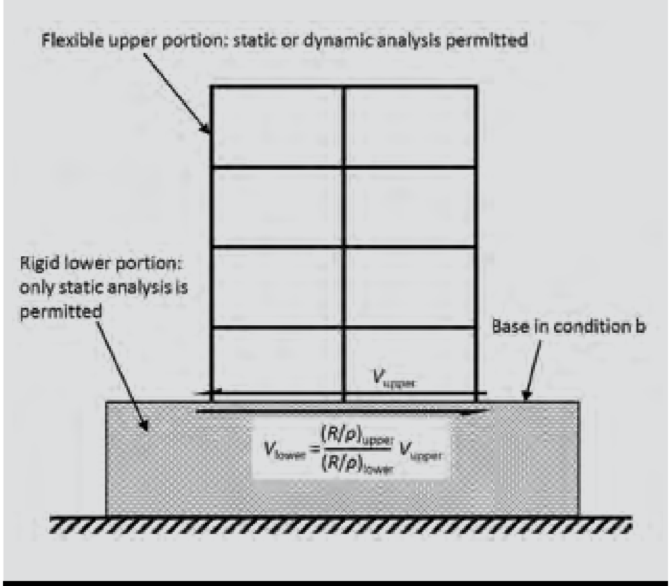

Figure 6. Two-stage analysis procedure. Note: R = response modification coefficient; V = total design lateral force or shear at the base; ρ = redundancy coefficient.

Height limit for special steel plate shear walls

Steel special plate shear wall systems were first introduced in the 2005 editions of ASCE 7 and AISC 341. During the incorporation of the seismic design parameters and height limitations for the system into ASCE 7-05 Table 12.2-1, the inclusion of this system in the permitted height increase of ASCE 7-05 section 12.2.5.4 was overlooked. This modification includes these systems in the permitted height increase of ASCE 7-10 section 12.2.5.4.

Steel ordinary and intermediate moment frames

Steel ordinary moment-frame construction has been used for many years for tall single-story buildings, including mill buildings, aircraft maintenance and assembly structures, and similar applications. ASCE 7-05 prohibited the use of ordinary and intermediate moment frames in higher seismic design categories for many of these structures. New exceptions[4] are added for SDC D and E ordinary and intermediate moment frames. The following important items are worth noting:

- To allow unlimited height, the sum of the dead and equipment loads cannot be greater than 20 lb/ft^2 (1000 Pa).

- The exterior wall weight must include the weight of exterior columns.

- For the case where cranes or other equipment is not self-supporting for all loads (that is, supported for vertical loads and/or laterally braced by columns that are part of or stabilized by ordinary or intermediate moment frames), the operating weight must be treated as fully tributary (100%) to either the adjacent exterior wall when located in an exterior bay or to the roof when located in an interior bay. The tributary area used for weight distribution must not exceed 600 ft^2 (56 m^2). Weights in exterior bays can also be tributary to the roof, if desired.

Flexible diaphragm condition

ASCE 7-05 section 12.3.1.1 set forth conditions under which certain diaphragms may be considered flexible for the purposes of lateral force distribution. The 2006 IBC section 1613.6.1 modified this ASCE 7-05 section to add one set of other conditions, the satisfaction of which would qualify a diaphragm as flexible. This modification was continued in the 2009 IBC. A modified set of the conditions included in this IBC modification is now part of ASCE 7-10.

The conditions in section 12.3.1.1 item c are based on results from the *Shake Table Tests of a Two-Story Woodframe House*,[18] which showed that for regular, light-framed, wood-diaphragm buildings, treating the diaphragms as flexible gives a better match with full-sized experimental tests. This research showed by full-scale tests that a thin, lightweight, nonstructural cellular concrete or gypsum topping does not appreciably change the stiffness of a wood diaphragm. Requiring separate shear wall lines to meet the drift criterion is a recommendation.[18] This ensures that the vertical elements of the lateral-force-resisting system are substantial enough to share load on a tributary basis.

Horizontal structural irregularities

In ASCE 7-05 Table 12.3-1, torsional as well as extreme torsional irregularity were defined in terms of the maximum story drift computed including accidental torsion. However, classification of torsional irregularity should not be iterative. Therefore clarification is now provided that it is accidental torsion with the torsional amplification factor A_x equal to 1.

The revised definition of nonparallel systems irregularity clearly indicates that it exists only where the vertical elements are not parallel to the major orthogonal axes. In other words, being parallel to the major orthogonal axes is sufficient to eliminate the irregularity. The ASCE 7-05 text of “parallel to or symmetric about” was sometimes misread to require that the system be both parallel to *and* symmetric about the major orthogonal axes. By that reading, **Fig. 7** has a nonparallel system irregularity. By the revised ASCE 7-10 definition, it does not.

Vertical structural irregularities

In ASCE 7-05, an in-plane discontinuity in vertical lateral-force-resisting element irregularity was defined to exist where an in-plane offset of the lateral-force-resisting elements was greater than the length of those elements or there existed a reduction in stiffness of the resisting element in the story below. The requirement of an in-plane offset to be greater than the length of an element was unconservative. On the other hand, there are many cases in which a lateral-force-resisting element may have a reduction in stiffness in the story below without causing an in-plane discontinuity. Thus, the definition of *vertical structural irregularity type 4* has been revised[4] to “in-plane discontinuity in vertical lateral force-resisting element is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on a supporting beam, column, truss, or slab.”

Redundancy provisions

The definition of height-to-length ratio of shear walls and wall piers has been clarified[4] for the purpose of determining the redundancy coefficient ρ. Wall height is from the top of a floor to the underside of the horizontal framing

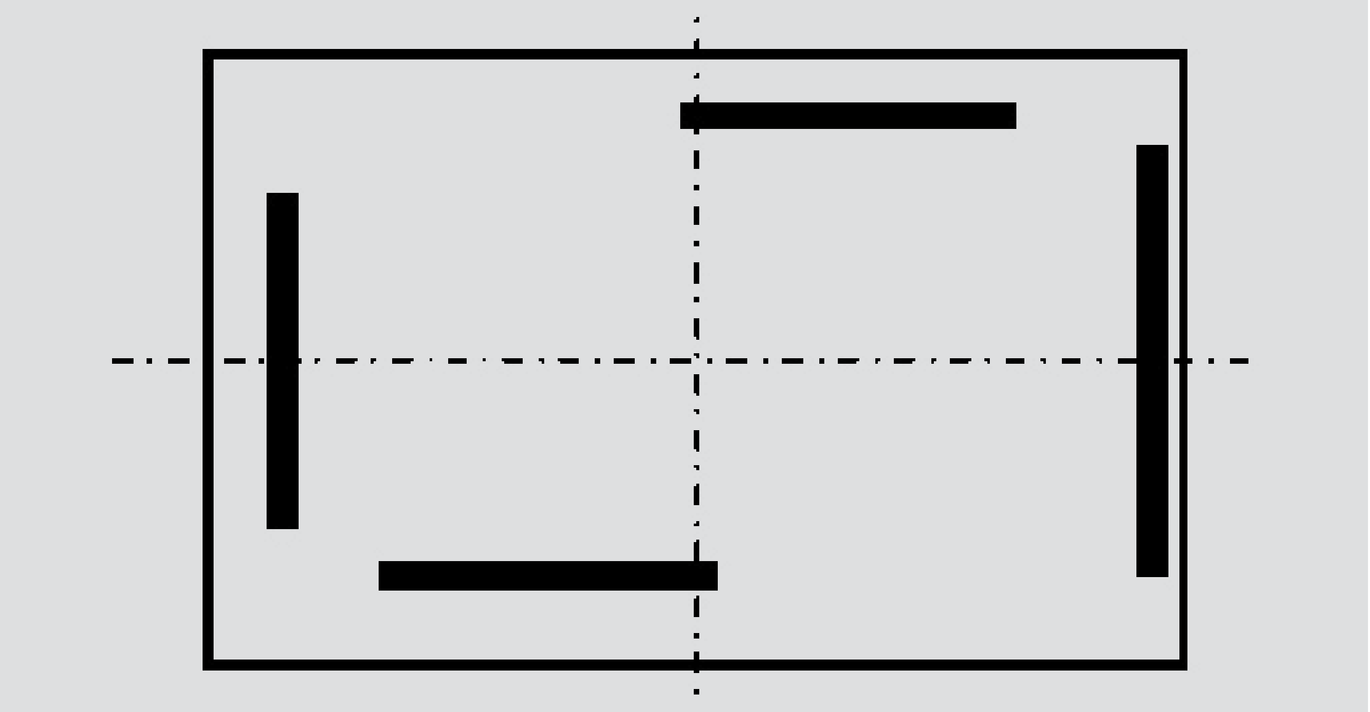

Figure 7. Asymmetrical seismic-force-resisting systems.

for the floor above, rather than to the top of the floor above (**Fig. 8**). Plywood shear walls that are 4 ft (1.2 m) long are thus sufficient to produce a redundancy factor of one for top-of-floor to top-of-floor height exceeding 8 ft (2.4 m), provided the wall height does not exceed 8 ft.

Increase in forces due to irregularities for seismic design categories D through F

The following changes have been made in ASCE 7-10 section 12.3.3.4 (underlined text indicates additions; struck-out text indicates deletions):

> For structures assigned to Seismic Design Category D, E or F and having a horizontal structural irregularity of Type 1a, 1b, 2, 3, or 4 in Table 12.3-1 or a vertical structural irregularity of Type 4 in Table 12.3-2, the design forces determined from Section ~~12.8.1~~ 12.10.1.1 shall be increased 25 percent for the following elements of the seismic force resisting system:
>
> 1. Connections of diaphragms to vertical elements and to collectors.
>
> 2. Collectors and their connections, including connections to vertical elements, of the seismic force-resisting system ~~and to connections of collectors to the vertical elements~~.
>
> Collectors and their connections also shall be designed for these increased forces unless they are designed for the load combinations with overstrength factor of Section 12.4.3.2, in accordance with Section 12.10.2.1.
>
> EXCEPTION: Forces calculated using the seismic load effects including overstrength factor of Section 12.4.3 need not be increased.

As can be seen, the ASCE 7-05 requirement concerning increases in forces due to irregularities for SDC D through F has been simplified by presenting the exception as such. The change also corrects the reference to the equivalent lateral force base shear in ASCE 7-05 section 12.8.1 (and, by implication, the corresponding vertical distribution) and refers to the diaphragm design force in ASCE 7-10 section 12.10.1.1 instead.

Conditions where redundancy factor ρ is 1.0

The redundancy factor ρ can now[4] be taken equal to 1.0 in the design of structural walls for out-of-plane forces, including their anchorage. The purpose of the redundancy factor was to penalize vertical seismic-force-resisting systems, such as shear walls in-plane, for lack of structural redundancy. The intent was not to penalize wall designs out-of-plane for nonredundant seismic-force-resisting systems.

Allowable stress increase for load combinations with overstrength

Where allowable stress design methodologies are used in conjunction with load combinations with overstrength,

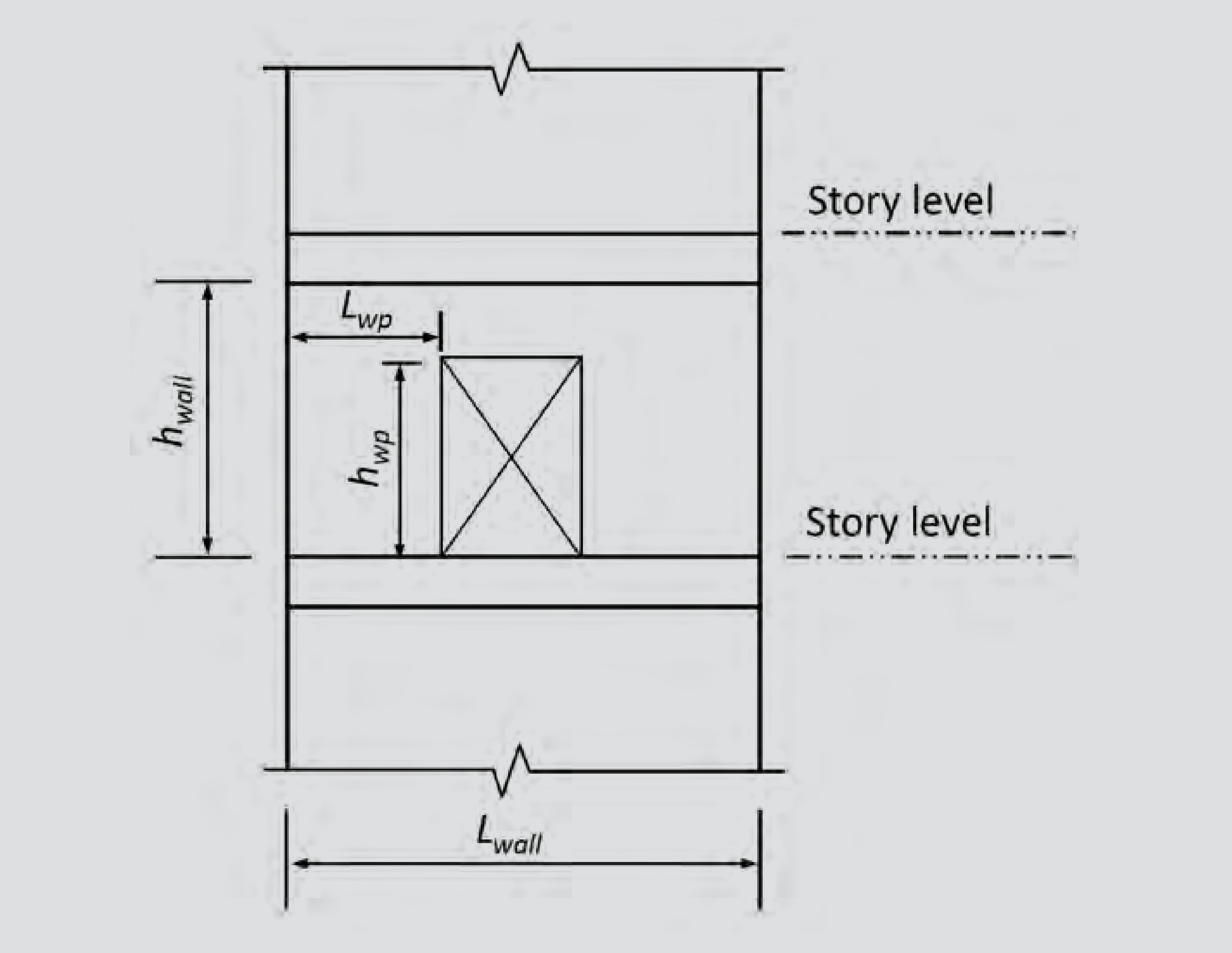

Figure 8: Height-to-length ratio of shear walls and wall piers. Note: shear wall height-to-length ratio = h_{wall}/L_{wall}; wall pier height-to-length ratio = h_{wp}/L_{wp}; h_{wall} = height of shear wall; h_{wp} = height of wall pier; L_{wall} = length of shear wall; L_{wp} = length of wall pier.

allowable stresses are permitted to be determined using an allowable stress increase of 1.2.

ASCE 7-05 used to require that "This increase shall not be combined with increases in allowable stresses or load combination reductions otherwise permitted by this standard or the material reference document except that combination with the duration of load increases permitted in American Forest and Paper Association's *National Design Specifications* (AF&PA NDS) is permitted."

This text has been changed in ASCE 7-10 to read, "This increase shall not be combined with increases in allowable stresses or load combination reductions otherwise permitted by this standard or the material reference document except for increases due to adjustment factors in accordance with AF&PA NDS."

Several adjustment factors for the design of wood construction can result in increases to the reference design values of the AF&PA NDS;[19] some examples are the flat use factor, repetitive member factor, buckling stiffness factor, and bearing area factor. These factors are material-dependent in much the same manner as the load duration factor.

Permitted analytical procedures

Two significant changes have been made to ASCE 7-10 Table 12.6-1, "Permitted Analytical Procedures." The table has been revised to eliminate unnecessary complexity and duplication. For SDC B and C buildings, the ASCE 7-05 table allowed all analysis procedures all the time. However, three rows in the upper portion of the table were used to communicate this. These three rows have been consolidated into one row. Also, in the first row applicable to SDC D, E, and F, "Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height" were exempted from dynamic analysis. This was redundant because the third row applicable to SDC D, E, and F exempted all light-framed buildings. This redundancy has been removed in ASCE 7-10.

The second significant change is the introduction of a new threshold for determining whether dynamic analysis is

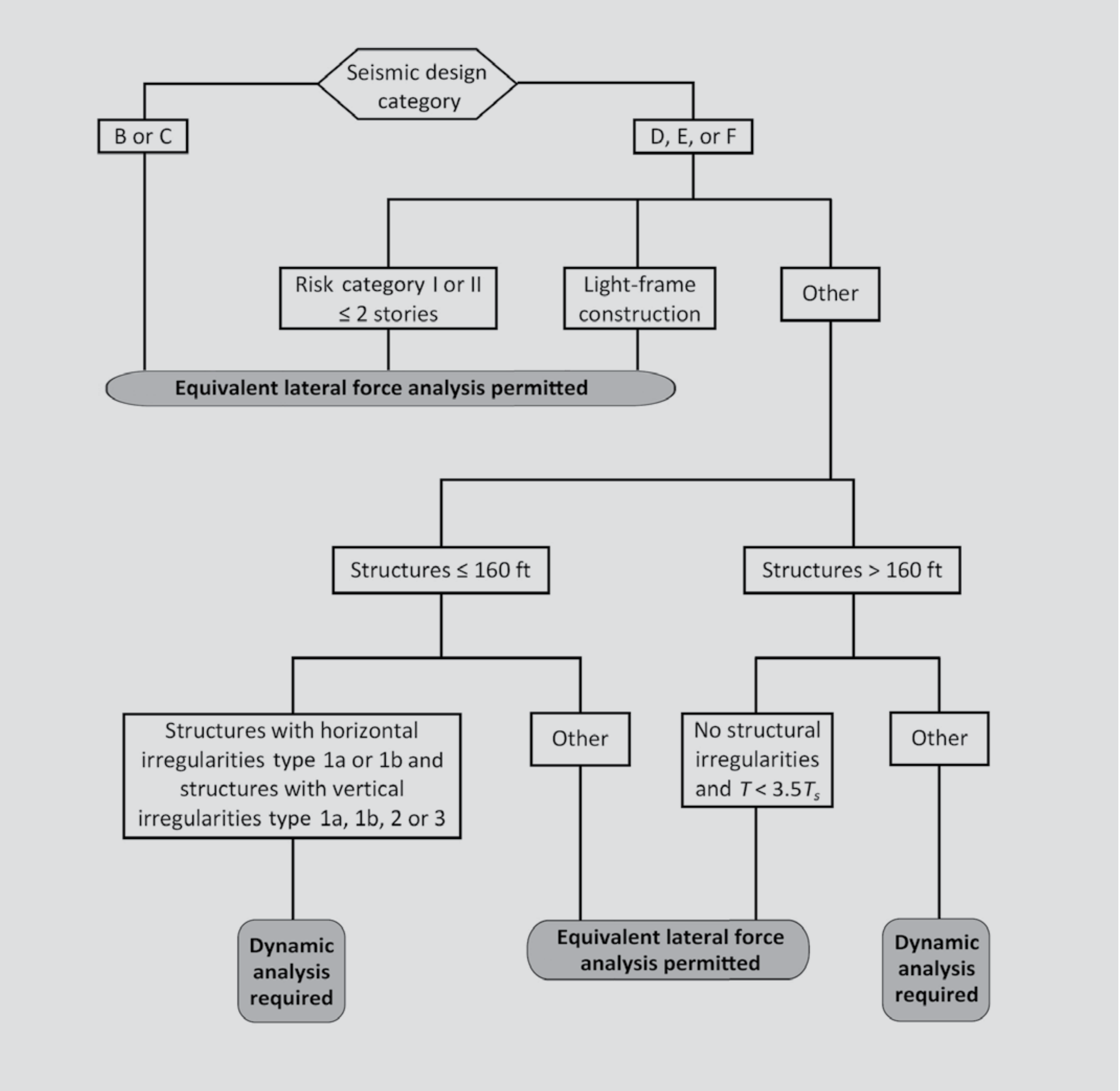

Figure 9. Flow chart illustrating when dynamic analysis is required by ASCE 7-10. Note: T = the fundamental period of the building; T_s = period at which the design spectrum transitions from its plateau to its descending branch, which varies with $1/T$. 1 ft = 0.305 m.

required. ASCE 7-10 establishes a new period-independent threshold of 160 ft (49 m), below which structures without certain irregularities are not required to be subject to dynamic analysis because higher mode effects are judged unlikely to be significant. Higher mode effects are still judged unlikely to be significant for regular structures exceeding 160 ft in height as long as the period remains less than the previous threshold of $3.5T_s$ (where T_S is the period at which the design spectrum transitions from its plateau to its descending branch, which varies with $1/T$ [$T_S = S_{D1}/S_{DS}$]; S_{D1} is design, 5%-damped, spectral response acceleration parameter at a period of 1 second; S_{DS} is design, 5%-damped, spectral response acceleration parameter at short periods), and dynamic analysis is not required.

Dynamic analysis is still required at times for shorter buildings with certain structural irregularities that tend to cause undesirable concentrations of inelastic displacements at certain locations. These structural irregularities are horizontal irregularities type 1a and 1b (torsional and extreme torsional), vertical irregularities type 1a and 1b (soft story and extreme soft story), vertical irregularity type 2 (weight/mass), and vertical irregularity type 3 (geometric). Equivalent lateral force procedure is not allowed for buildings with the listed 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. **Figure 9** is a flow chart to determine when a dynamic analysis is required by ASCE 7-10.

Effective seismic weight

What is required to be included in the effective seismic weight of a building as well as a nonbuilding structure is better defined. The following changes have been made in ASCE 7-10 section 12.7.2 (underlined text indicates additions; struck-out text indicates deletions):

> "The effective seismic weight, *W*, of a structure shall include the ~~total~~ dead load, as defined in Section 3.1, above the base and other loads above the base as listed below:
>
> 1. In areas used for storage, a minimum of 25 percent of the floor live load shall be included. ~~(floor live load in public garages and open parking structures need not be included).~~
>
> EXCEPTIONS:
>
> a. Where the inclusion of storage loads adds no more than 5% to the effective seismic weight at that level, it need not be included in the effective seismic weight.
>
> b. Floor live load in public garages and open parking structures need not be included.
>
> 5. Weight of landscaping and other materials at roof gardens and similar areas."

Items 2, 3, and 4 of ASCE 7-10 section 12.7.2 remain unchanged. A corresponding change has been made in ASCE 7-10 section 12.14.

Structural modeling

The applicability of required consideration of cracked section properties in concrete and masonry structures and panel zone deformations in steel moment frames has been clarified.[4] A new exception exempts structures with flexible diaphragms and type 4 horizontal structural irregularity from 3-D analysis requirement.

Table 2. Changes to ASCE 7-10 Table 12.8-2

Structure type	C_t	x
Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1	0.03	0.75
Steel buckling-restrained braced frames	0.03	0.75

Note: C_t = building period coefficient; x = level under consideration.

Minimum design base shear

The minimum design base shear of $0.044S_{DS}I_eW$ (where I_e is importance factor), applicable for SDC B through F, was part of ASCE 7-02[20] and the 2000[21] and 2003[22] IBC. However, when the third (constant-displacement) branch, starting at the long-period transition period T_L, was added to the design response spectrum of ASCE 7-05, this minimum base shear was deleted in favor of just 1% of weight, which is a structural integrity minimum, applicable irrespective of SDC. The basis was that the long-period structure was now being directly addressed by the constant-displacement branch of the design spectrum so that there was no need for an arbitrary minimum value.

In the course of the ATC-63 project,[12] a large number of ordinary as well as special moment frames of concrete were analyzed by state-of-the-art dynamic analysis procedures, each frame under a large number of pairs of earthquake ground motions. The analyses disturbingly showed story mechanisms forming even in the special moment frames, which satisfied the strong column-weak beam requirement, early into earthquake excitations. After considerable discussion, these frames, designed in accordance with ASCE 7-05, were redesigned in accordance with ASCE 7-02 instead, in effect reinstating the minimum design base shear requirement of $0.044S_{DS}I_eW$. The aforementioned problem went away, leading to the inescapable conclusion that removal of the minimum base shear had been a mistake. ASCE processed supplement no. 2 to ASCE 7-05, reinstating this minimum design base shear. Supplement no. 2 was adopted by the 2009 IBC. ASCE 7-10 has now incorporated supplement no. 2 in its body (**Fig. 10** shows design response spectrum with minimum base shears).

Equation (12.8-5) to determine seismic response coefficient C_s has been changed in ASCE 7-10 as shown:

$$C_s = \cancel{0.01}\ \underline{0.044S_{DS}I_e \geq 0.01} \qquad \text{(Eq. 12.8-5)}$$

Approximate fundamental period

Table 2 shows the changes that have been made in ASCE 7-10 Table 12.8-2. The longer predicted periods represented by the building period coefficient C_t of 0.03 for steel eccentrically braced frames are appropriate where significant eccentricities exist, such as those designed in accordance with AISC 341-10. The added wording provides clarification and ensures that significant eccentricities exist.

The steel buckling restrained braced frame system was first approved for the 2003 NEHRP provisions. The values for the approximate period parameters C_t and x were also approved. Somehow these parameters were not carried forward into ASCE 7-05. These two factors were in appendix R of AISC 341-05. These have been removed from AISC 341-10 in view of this change in ASCE 7-10.

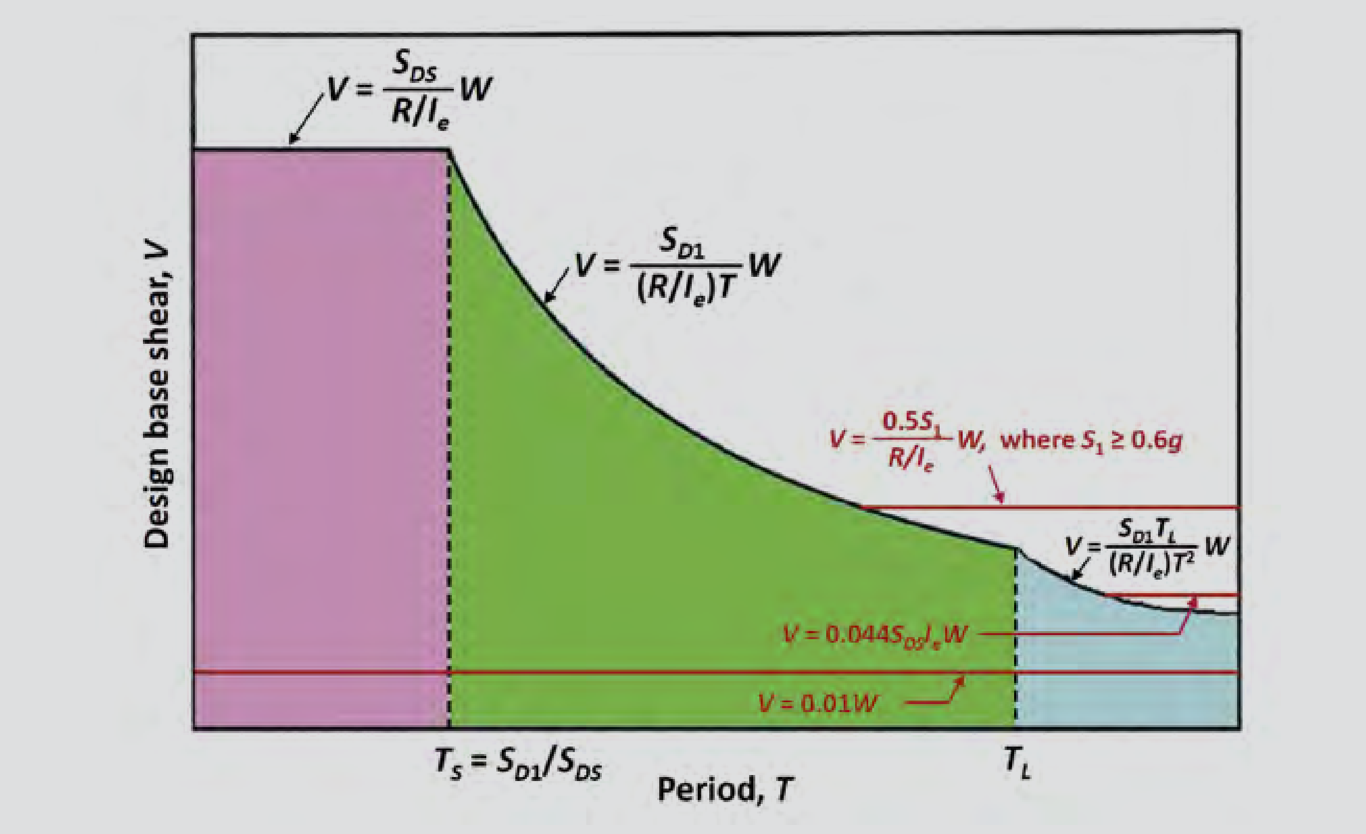

Figure 10. ASCE 7-10 design response spectrum. Note: g = acceleration due to gravity; I_e = importance factor; R = response modification coefficient; S_1 = mapped MCE_R, 5%-damped, spectral response acceleration parameter at a period of 1 second; S_{D1} = design, 5%-damped, spectral response acceleration parameter at a period of 1 second; S_{DS} = design, 5%-damped, spectral response acceleration parameter at short periods; T_L = long-period transition period as defined in ASCE 7-10 section 11.4.5; T_s = period at which the design spectrum transitions from its plateau to its descending branch, which varies with $1/T$; V = total design lateral force or shear at the base; W = effective seismic weight of the building.

Approximate period formula based on number of stories

In defining the applicability of ASCE 7-10 Eq. (12.8-8), the 10 ft (3 m) minimum story height has been revised such that it is now an average story height. Let us take the hypothetical case of a frame structure with a minimum story height of 9 ft (2.7 m) and average story height of 10.5 ft (3.2 m). ASCE 7-05 Eq. (12.8-8) would not have been applicable to this situation. ASCE 7-10 Eq. (12.8.8), however, is applicable.

Amplification of accidental torsional moment

In section 12.8.4.3, the ASCE 7-05 exception that reads, “The accidental torsional moment need not be amplified for structures of light-frame construction,” has been deleted.[4] Where wood-frame diaphragms are designed as rigid diaphragms (one example is diaphragms in open-front structures), amplification of torsion should apply. Also, it is now explicitly stated that the torsional amplification factor A_x shall not be less than one because it is possible for A_x to be less than one per ASCE 7-10 Eq. (12.8-14).

Story drift determination

Story drift limits (and not the computation of story drift demand Δ) are the focus of ASCE 7-10 section 12.12.1. Determination of story drift demand is treated in ASCE 7-10 section 12.8.6. Therefore, to provide a distinct separation between limit and demand, the last sentence in ASCE 7-05 section 12.12.1 that discusses determination of story drift when horizontal irregularity type 1a or 1b is present is moved to ASCE 7-10 section 12.8.6. Also, ASCE 7-10 section 12.8.7 (“P-Delta Effects”) references ASCE 7-10 section 12.8.6 and not section 12.12.1. The intent is not to limit Δ by taking displacements at the centers of mass for P-Δ (where P is vertical design load) computation when horizontal irregularity type 1a or 1b is present.

Many computer programs can explicitly provide drift ratios; however, such programs often do not use the same vertically aligned points to compute these ratios, thus yielding inaccurate measures of drift. A sentence was added in the first paragraph of ASCE 7-10 section 12.8.6 to permit vertical projections of points when centers of mass do not align vertically. Vertically aligned points are also called for in the second paragraph, which applies to structures assigned to SDC C and above and having torsional or extreme torsional irregularities. In these cases, torsion must be included in deflection computation so that drifts are based on diaphragm-edge deflections, rather than deflections at the centers of mass.

Minimum base shear for computing drift

ASCE 7-10 section 12.8.6.1 has been revised as shown (underlined text indicates addition): "The elastic analysis of the seismic force-resisting system <u>for computing drift</u> shall be made using the prescribed seismic design forces of Section 12.8.

Exception: Eq. 12.8-5 need not be considered for computing drift."

The 1997 *Uniform Building Code* (UBC)[23] exempted the minimum base shear of $0.11C_aIW$ (where C_a is seismic coefficient, and I is importance factor) from drift computation. This was not adopted by ASCE 7-02, ASCE 7-05, or the first four editions of the IBC.[21,22,23] Now the exemption has been reinstated. This change is significant when it comes to the design of tall buildings.

Tall buildings are drift controlled rather than strength controlled. The design of many tall buildings, irrespective of seismic design category, is likely to be governed, in the absence of this exemption, by drift computed under the minimum design base shear given by ASCE 7-10 Eq. (12.8-5). This is considered unjustified because this minimum design base shear is essentially a minimum strength requirement. The near-fault minimum, as given by ASCE 7-10 Eq. (12.8-6), has a physical basis and is not exempt.

P-Δ effects

The importance factor I_e has now been included in the denominator of the expression for the stability coefficient θ, ASCE 7-10 Eq. (12.8-16). In the 2003 NEHRP provisions, the importance factor is included in the stability coefficient, as it is in the 2009 NEHRP provisions.[24]

Combined response parameters in modal response spectral analysis

ASCE 7-10 section 12.9.3 has been modified as follows:

> The value for each parameter of interest calculated for the various modes shall be combined using ~~either~~ the square root of the sum of the squares (SRSS) method, ~~or~~ <u>the complete quadratic combination (CQC) method</u>, the complete quadratic combination method ~~(CQC)~~ <u>as modified by</u> ~~in accordance with~~ ASCE 4 <u>(CQC-4), or an approved equivalent approach</u>. The CQC <u>or the CQC-4</u> method shall be used for each of the modal values or where closely spaced modes ~~that~~ have significant cross-correlation of translational and torsional response.

The CQC modal response combination method, as it is presented in ASCE 4-98 *Seismic Analysis of Safety-Related Nuclear Structures*,[25] varies slightly from the classical method as implemented by various commercially available structural analysis software packages.

Scaling of drifts in modal response spectral analysis

Provision has been added for scaling of drifts where the near-fault minimum base shear equation (ASCE 7-10 Eq. [12.8-6]) governs. Where the combined response for the seismic base shear V_t is less than $0.85C_sW$, where C_s is determined in accordance with ASCE 7-10 Eq. (12.8-6), drifts are required to be multiplied by $0.85C_sW/V_t$.

Diaphragm and collector design forces

In ASCE 7-10 section 12.4.3.1, it has been clarified that diaphragm design forces are earthquake load effects Q_E as used in the load combinations of ASCE 7-10 section 12.4. Equation numbers in ASCE 7-10 section 12.10.1.1 have been added to the expressions for the minimum and maximum forces facilitating reference.

ASCE 7-10 section 12.10.2.1 has been revised so that three checks need to be made in determining design forces for collector elements and their connections. ASCE 7-05 did not require consideration of the diaphragm design forces of ASCE 7-05 section 12.10.1-1.

For structures assigned to SDC C through F, design forces for collector elements and their connections are the maximum of the following (**Fig. 11**):

- forces determined from the overall building analysis under the design-based shear V amplified by the overstrength factor of ASCE 7-10 section 12.4.3, that is Ω_0Q_E
- forces determined from ASCE 7-10 Eq. (12.10-1), diaphragm design force at floor level x F_{px} amplified by the overstrength factor of ASCE 7-10 section 12.4.3, that is Ω_0F_{px}
- forces determined from ASCE 7-10 Eq. (12.10-2), minimum value of F_{px} that can be used in design $F_{px,min}$ without any overstrength factor

The maximum collector forces determined from the previous bullets need not exceed those obtained from ASCE 7-10 Eq. (12.10-3), maximum value of F_{px} that need not be exceeded in design $F_{px,max}$ without overstrength factor. A

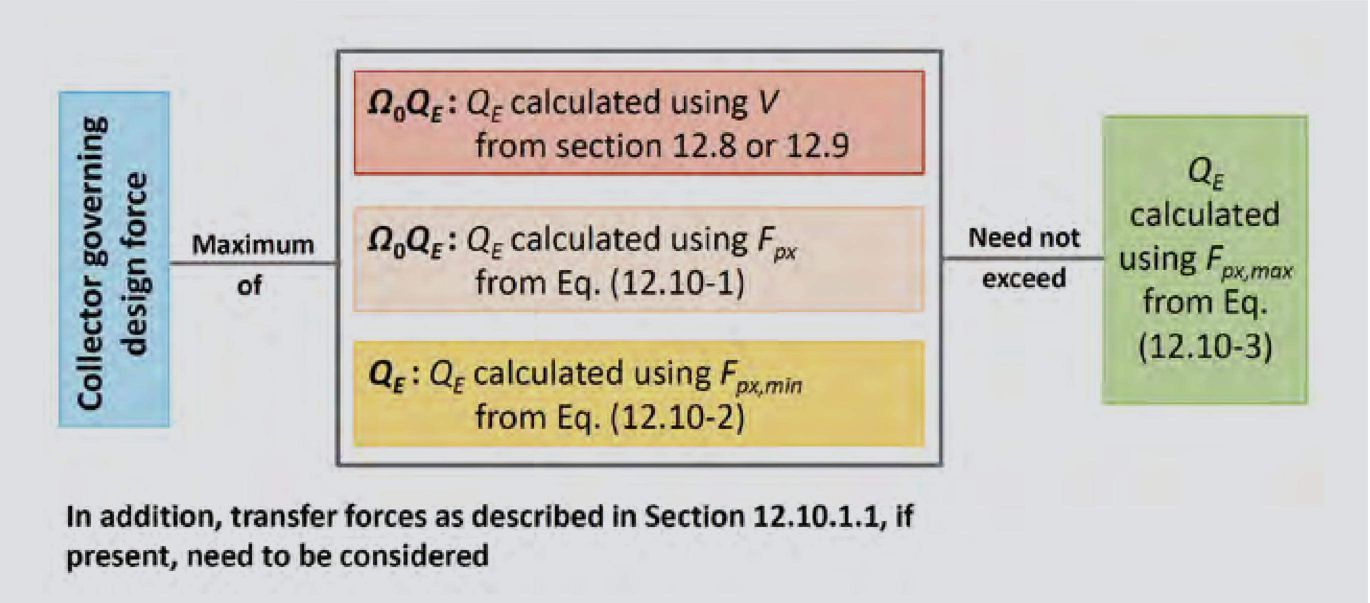

Figure 11. Collector design force of ASCE 7-10. Note: F_{px} = diaphragm design force at floor level x; $F_{px,max}$ = value that F_{px} need not exceed; $F_{px,min}$ = minimum value of F_{px} that can be used in design; Q_E = effect of horizontal seismic (earthquake induced) forces; V = total design lateral force or shear at the base; Ω_0 = overstrength factor.

change proposing that the overstrength be applied to $F_{px,max}$ has been submitted for ASCE 7-16, which is under development. This proposed change does have merit. The reader should be cautioned not to use this particular provision of ASCE 7-10 as printed in the document. It should instead be used with the overstrength factor applied to $F_{px,max}$, as proposed for ASCE 7-16.

Design for out-of-plane forces

In ASCE 7-05, there was no logical path for out-of-plane structural wall forces to be included in the seismic load combinations because they were not specifically defined as either V or seismic force acting on a component of a structure F_p. The term Q_E, as identified under ASCE 7-05 section 12.4.2.1, is derived only from V or F_p. The out-of-plane structural wall force of $0.4S_{DS}I$ in ASCE 7-05 section 12.11.1 was not labeled as F_p. Thus how out-of-plane forces entered the load combination equations remained technically unresolved. This is resolved[4] by stating F_p equals $0.4S_{DS}I_e$ times the weight of the structural wall with a minimum force of 10% of the weight of the structural wall.

Structural separation and property line setback

Structural separation and setback provisions were included in the 1997 UBC as well as the 2000 and the 2003 editions of the IBC. However, when the 2006 IBC was being developed, it was decided to delete much of the structural provisions from the code itself and incorporate them only through reference to ASCE 7-05. The building separation provisions were deleted, overlooking the fact that ASCE 7-05 did not include any such requirements. This error was rectified by having the building separation provisions included in the 2009 IBC by way of a modification to ASCE 7-05. The modification has now been incorporated into ASCE 7-10.

The provisions are the same as those included in the 2003 and the 2000 IBC, where the separation between two adjacent buildings needs to be adequate to accommodate the maximum inelastic floor displacements of the two buildings. The maximum inelastic floor displacement δ_M is computed as:

$$\delta_M = \frac{C_d \delta_{max}}{I_e}$$

where

δ_{max} = maximum elastic displacement that occurs anywhere in a floor from the application of the design base shear to the structure

The maximum elastic displacement δ_{max} includes the effects of translation plus rotation due to inherent as well as accidental torsion. δ_{max} is different from the elastic floor displacement δ_{xe}, which is determined at the center of mass of the floor and is used in ASCE 7-10 Eq. (12.8-15) to compute inelastic floor deflection.

The maximum inelastic floor displacements from adjacent buildings are combined by the square root of the sum of the squares method to determine the "distance sufficient to avoid damaging contact." Where a structure adjoins a property line not common to a public way, the structure also needs to be set back from the property line by at least δ_M (**Fig. 12**).

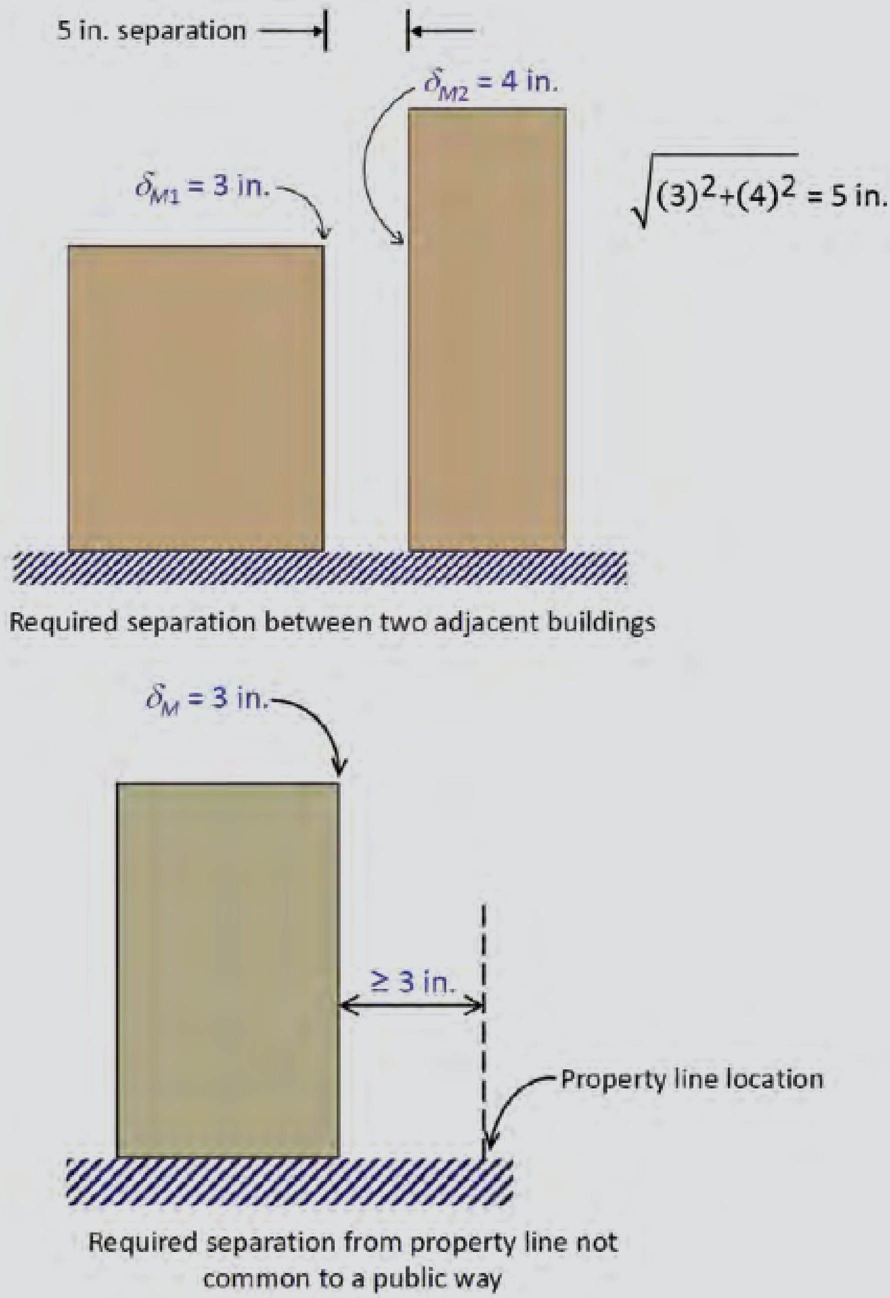

Figure 12. Building separation requirements of ASCE 7-10. Note: δ_M = maximum inelastic response displacement, considering torsion; δ_{M1} = δ_M at roof level of shorter building; δ_{M2} = δ_M at the same height of taller neighboring building. 1 in. = 25.4 mm.

The importance factor is included in the maximum elastic displacement δ_{max} through the computation of base shear. Thus, when $C_d\delta_{max}$ is divided by I_e, the effect of building occupancy is canceled out.

Anchorage of structural walls and transfer into diaphragms

Several significant changes have been made in the provisions concerning the design force for the anchorage between walls and floor or roof diaphragms providing lateral support. ASCE 7-05 section 11.7.5 contained provisions for concrete and masonry walls assigned to SDC A. That section, with modifications, is now section 1.4.5 in ASCE 7-10. The new location is a clear indication that the requirements are basic structural integrity requirements. The 280 lb/ft (4.09 kN/m) minimum requirement has been replaced by 0.2 times the weight of wall tributary to the connection, but not less than 5 lb/ft^2 (240 Pa). The requirements now apply to all walls, not just concrete and masonry walls.

ASCE 7-05 sections 12.11.2 “Anchorage of Concrete or Masonry Structural Walls” (in structures assigned to SDC

B through F [Fig. 12]) and 12.11.2.1 “Anchorage of Concrete or Masonry Structural Walls to Flexible Diaphragms” (in structures assigned to SDC C through F [Fig. 12]) have been replaced in ASCE 7-10 by the newly titled sections 12.11.2 “Anchorage of Structural Walls and Transfer of Design Forces into Diaphragms” and 12.11.2.1 “Wall Anchorage Forces,” both of which are applicable to structures assigned to SDC B through F (**Fig. 13**). The changes improve the organization of the anchorage provisions. Similar revisions have been made in section 12.14 for the simplified seismic design method.

There are several substantive changes to the anchorage provisions. First, there is no longer any distinction between concrete and masonry walls and all walls. Second, the lower-bound anchorage force of $0.10W_p$ (where W_p is the weight of wall tributary to anchor; 280 lb/ft [4.09 kN/m] in the case of concrete and masonry walls) has been replaced by a minimum force of $0.2k_aI_eW_p$ (**Fig. 14**). The multiplier k_a increases from 1.0 to 2.0 as the span of a flexible diaphragm L_f (**Fig. 15**) increases from 0 to 100 ft (30 m) or more. This span is considered to be zero for a rigid diaphragm, yielding a k_a of 1.0. This change results in rather significant increases in the anchorage design force for taller walls in areas of moderate to low seismic hazard (where S_{DS} values are moderate to low). Third, the anchorage design force for walls supported by flexible diaphragms used to be twice that for walls supported by rigid diaphragms. ASCE 7-10 provides a gradual increase in anchorage design force through the multiplier k_a. In a further important change, where the anchorage is not located at the roof and all diaphragms are not flexible, the anchorage design force given by ASCE 7-10 Eq. (12.11-1) may be reduced through multiplication by $(1 + 2z/h)/3$, where z is the height of the anchor above the base of the structure and h is the height of the roof above the base. This is consistent with the variation in seismic design force for nonstructural components attached to a building along the height of the building, as given in ASCE 7-10 section 13.3.1.

Members spanning between structures

ASCE 7-05 provisions did not specifically address the situation where a seismic separation exists between two buildings but the gravity system is not separate. Large relative movements of the seismically separate building portions may lead to loss of gravity support for members that bridge between the two portions unless supports are designed to accommodate such displacements. Five requirements are given in ASCE 7-10 for conservatively estimating these movements.

Openings or reentrant building corners

Perforated shear walls are permitted in the AF&PA *Special Design Provisions for Wind and Seismic*,[26] which is referenced in the exception to ASCE 7-05 section 12.14.7.2. AISI S213 *North American Standard for Cold-Formed*

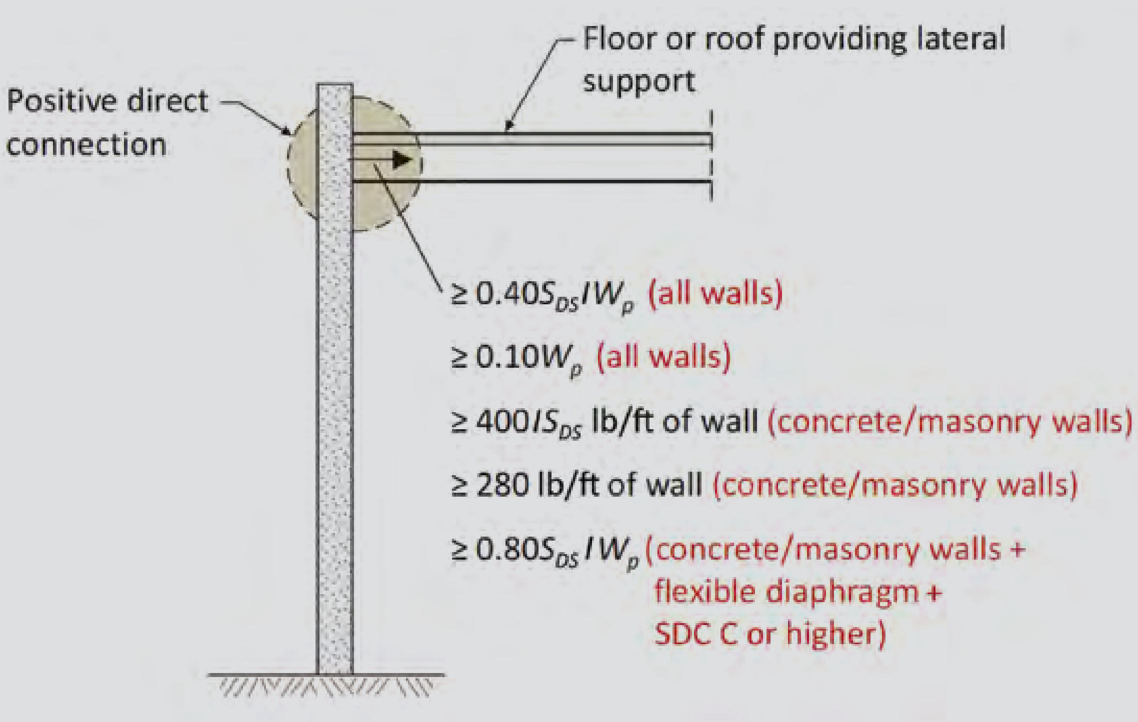

Figure 13. Anchorage of structural walls–ASCE 7-05 requirements. Note: I = importance factor; S_{DS} = design, 5% damped, spectral response acceleration parameter at short periods; W_p = weight of wall tributary to anchor.

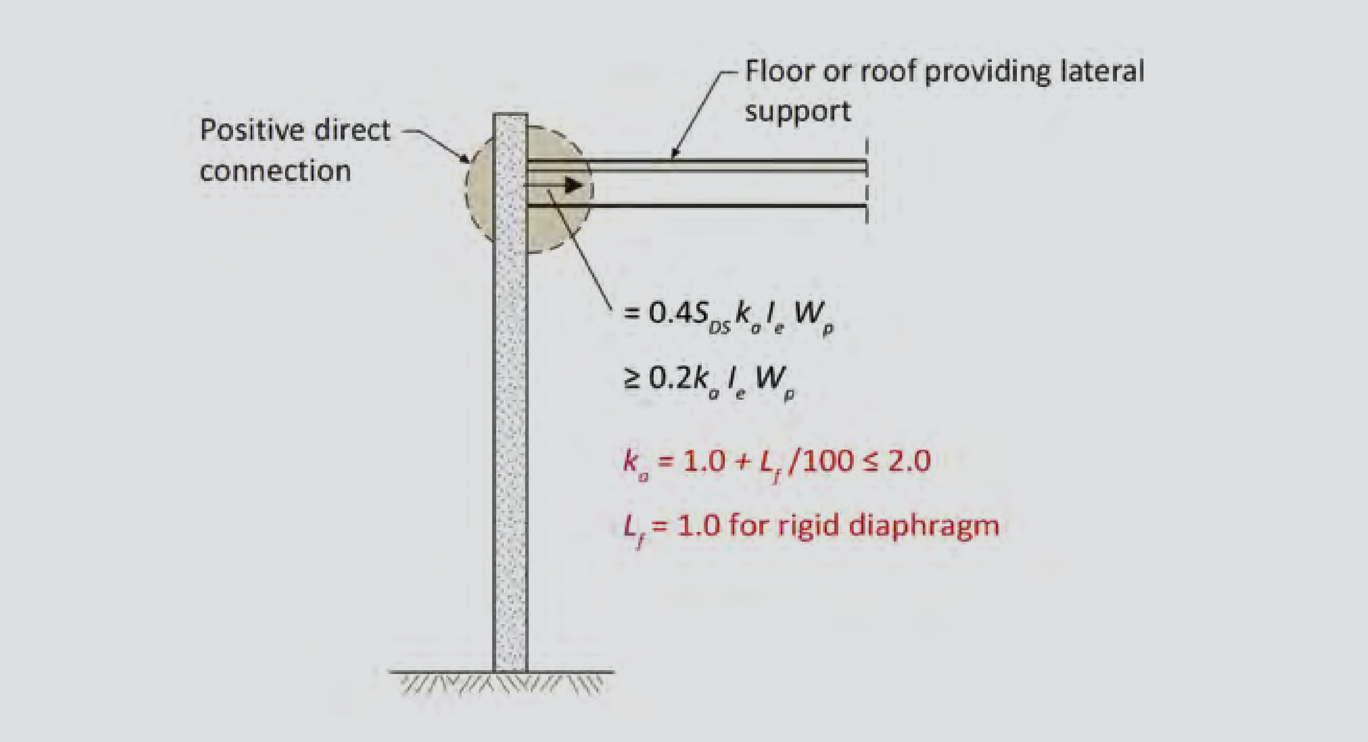

Figure 14. Anchorage of structural walls–ASCE 7-10 requirements. Note: I_e = importance factor; k_a = multiplier for diaphragm flexibility; L_f = span of a flexible diaphragm that provides the lateral support for the wall; S_{DS} = design, 5%-damped, spectral response acceleration parameter at short periods; W_p = weight of wall tributary to anchor.

Steel Framing—Lateral Design[27] has now been developed for a similar cold-formed steel system called *Type II shear walls*. The exception to ASCE 7-10 section 12.14.7.2 has been expanded to recognize Type II shear walls that are in compliance with AISI S213, based on testing that has shown that a portion of the shear forces can be transferred through the steel framing around openings.

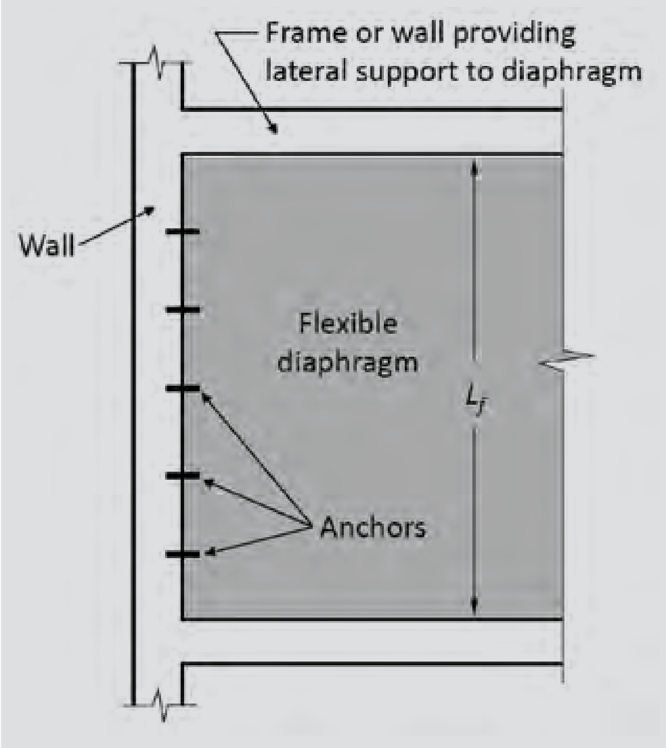

Figure 15. Anchorage of walls to flexible diaphragm. Note: L_f = span of a flexible diaphragm that provides the lateral support for the wall.

Other structural changes

Changes in chapters 16 through 22 are discussed in this section.

Three-dimensional seismic response history analysis

Studies of 50 M6.5 to M7.9 ground motions indicated that the maximum direction of ground motion is slightly less than the SRSS of the two components by a factor of approximately 1.16. In view of this, the phrasing of the ASCE 7-05 language is simplified in ASCE 7-10 section 16.1.3.2 by replacing "10% less than 1.16 times the MCE response spectrum" with "the MCE spectrum," resulting in an effective 1.0 multiplier [(0.9)(1.16) ≈ 1.0].

For sites within approximately 3 mi. (5 km) of an active fault that controls the ground-motion hazard, the near-field strong-motion database indicates that the fault-normal direction is (or is close to) the direction of maximum ground motion for periods around 1.0 second and greater). In this case, the two horizontal components of a selected record should be transformed so that one component is the motion in the fault-normal direction and the other component is the motion in the fault-parallel direction. Scaling so that the average fault-normal component response spectrum is at the level of the MCE response spectrum ensures that the fault-normal components will not be underestimated, which would happen if the SRSS rule were applied at short distances. The same scale factor selected for the fault-normal component of a given record is to be used for the fault-parallel component as well.

Response parameters from linear response history analysis

While force-related response parameters, such as bending moments, shear forces, story shears, and base shear, resulting from linear response history analysis are to be multiplied by I_e/R, the displacement-related response quantities, such as lateral displacements, are to be multiplied by C_d/R (ASCE 7-10 section 16.1.4).

Horizontal shear distribution in linear response history analysis

Consideration of accidental torsion for linear response history analysis (ASCE 7-10 section 16.1.5) has been made consistent with that for modal response spectral analysis (ASCE 7-10 section 12.9.5). The distribution of horizontal shear is required to be in accordance with ASCE 7-10 section 12.8.4, which requires that the seismic design story shear V_x be distributed to the various vertical elements of the seismic-force-resisting system in the story under consideration based on the relative lateral stiffnesses of the vertical resisting elements and the diaphragm. Amplification of torsion in accordance with ASCE 7-10 section 12.8.4.3 is not required where accidental torsion effects are included in the dynamic analysis model.

Values of shear wave velocity and shear modulus for soil-structure interaction analysis

ASCE 7-05 Table 19.2-1 used single values of shear wave velocity and shear modulus reduction factors (from values at small strains to values at large strains), which failed to account for differences in shear strain associated with soils having different stiffnesses. A revised table was developed for FEMA 356 *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*[28] to correct that error. The revised table was adopted into the 2009 NEHRP provisions and is now the revised Table 19.2-1 in ASCE 7-10.

Foundation damping factor in soil-structure interaction analysis

ASCE 7-05 Fig. 19.2-1 could not be reproduced from the source articles supposedly used to derive it. To remedy this, a substitute figure was developed in the ATC-55 project, the primary product of which was the FEMA 440 report *Improvement of Nonlinear Static Analysis Procedures,*.[29] The ATC-55 project was conducted to develop guidelines for improved application of the coefficient method, as detailed in FEMA-356, the capacity spectrum method as detailed in the ATC-40 report, *Seismic Evaluation and Retrofit of Concrete Buildings*[30], and improved application of nonlinear static analysis procedures in general. The figure was used in FEMA-440. The figure was incorporated in the 2009 NEHRP provisions and is now included in ASCE 7-10 as revised Fig. 19.2-1.

Deterministic lower limit on MCE_R response spectrum from site response analysis

Figure 21.2-1 in ASCE 7-05 was not correct because it did not show the ramp building up to the flat top or the segment beyond the long-period transition period. The revised Fig. 21.2-1 in ASCE 7-10 corrects these omissions.

Design acceleration parameters from site-specific ground motion procedures

ASCE 7-10 section 21.4 specifies the approach to determine design acceleration parameters S_{DS} and S_{D1} when the site-specific procedure is used. The values of S_{DS} and the design, 5%-damped, spectral response acceleration parameter at a period of 1 second S_{D1} are important in the determination of the following:

- seismic design category (S_{DS} and S_{D1})
- load combinations (S_{DS})
- out-of-plane wall and anchorage forces (S_{DS})
- coefficient for upper limit on calculated period C_u for upper bound on rationally computed period (S_{D1})
- nonstructural design force (S_{DS})
- scaling of results of modal response spectral analysis (which refers to 85% of value given by equivalent lateral force procedure formulas, which use both S_{DS} and S_{D1})

It was never intended that the values of S_{DS} and S_{D1} given by ASCE 7-10 section 21.4 be used in the determination of the equivalent lateral force procedure base shear by ASCE 7-10 section 12.8. Rather, the site-specific spectrum obtained using ASCE 7-10 chapter 21 should be used for the latter purpose. Changes have been made to clarify this intent by specifying the appropriate modifications to ASCE 7-10 Eq. (12.8-3) and (12.8-4) when using the site-specific spectrum approach. The changes further clarify that the parameter S_{DS} is permitted to be used in ASCE 7-10 Eq. (12.8-2), (12.8-5), (15.4-1), and (15.4-3) and that the mapped value of S_1 is to be used in Eq. (12.8-6), (15.4-2), and (15.4-4).

Conclusion

Major revisions have taken place in the seismic design provisions from ASCE 7-05 to ASCE 7-10. The seismic hazard maps used in seismic design have undergone profound changes that are fourfold. These changes to the seismic maps are presented along with explanations as to why the changes were necessary and how they will affect seismic design results. The combined changes should not cause substantive differences in the seismic designs that result from ASCE 7-05 and ASCE 7-10, except in certain locations within the United States.

There are many other significant changes to the ASCE 7 seismic provisions. These include the following:

- major changes in the design force requirements for the anchorage between concrete, masonry, and other walls and diaphragms providing lateral support
- changes to Table 12.2-1 (the *R*-values table) and in the rules governing combinations of structural systems
- increased height limits for structural systems including special steel plate shear walls
- changes in approximate fundamental period for eccentrically braced frame and buckling-restrained braced frame systems
- significant changes in Table 12.6-1 Permitted Analytical Procedures
- permitting single-story industrial buildings with steel ordinary moment frames or intermediate moment frames to unlimited height in SDC D and E
- recognition of cold-formed steel special bolted moment frames and their inclusion in Table 12.2-1
- major enhancements introduced in chapter 13, "Nonstructural Components"
- significant enhancements made in chapter 15, "Nonbuilding Structures"

Major materials standards referenced from chapter 14 have been updated. Chapter 14 changes are excluded from the scope of this article because chapter 14 is not adopted by the IBC. Changes in chapters 13 and 15 are also excluded to avoid excessive length.

It is understood that not too many precasters are probably interested in as much detailed information as is provided, for instance, on the seismic ground motion maps. Still, a comprehensive approach to the changes has been chosen for this paper for two reasons. First, there are always a few who are interested in detailed background information and in changes affecting competing materials. Second, there are situations where knowledge of the background is useful to help make the right decision.

Significant Changes to the Seismic Load Provisions of ASCE 7-10: An Illustrated Guide[31] contains more extended discussion on every significant seismic change from ASCE 7-05 to ASCE 7-10.

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References

1. ASCE (American Society of Civil Engineers). 2005. *Minimum Design Loads for Buildings and Other Structures*. ASCE 7-05. Reston, VA: ASCE.

2. ICC (International Code Council). 2006. *International Building Code*. Falls Church, VA: ICC.

3. ICC. 2009. *International Building Code*. Falls Church, VA: ICC.

4. ASCE. 2010. *Minimum Design Loads for Buildings and Other Structures*. ASCE 7-10. Reston, VA: ASCE.

5. ICC. 2012. *International Building Code*. Country Club Hills, IL: ICC Publications.

6. Petersen, M. D., A. D. Frankel, S. C. Harmsen, C. S. Mueller, K. M. Haller, R. L. Wheeler, R. L. Wesson, Y. Zeng, O. S. Boyd, D. M. Perkins, N. Luco, E. H. Field, C. J. Wills, and K. S. Rustakes. 2008. *Documentation for the 2008 Update of the United States National Seismic Hazard Maps.* Open-file report 2008-1128. Reston, VA: U.S. Geological Survey.

7. EERI (Earthquake Engineering Research Institute). 2008. "Next Generation Attenuation Relationships." Special Issue, *Earthquake Spectra* 24 (1).

8. FEMA (Federal Emergency Management Agency). 1997. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures.* Washington, DC: FEMA.

9. FEMA. 2000. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. Washington, DC: FEMA.

10. FEMA. 2003. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. Washington, DC: FEMA.

11. Luco, N., B. R. Ellingwood, R. O. Hamburger, J. D. Hooper, J. K. Kimball, and C. A. Kircher. 2007. "Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States." In *Proceedings*, Structural Engineers Association of California (SEAOC) 2007 Convention, September, Squaw Creek, CA, SEAOC, Sacramento, CA.

12. FEMA. 2009. *Quantification of Building Seismic Performance Factors*. FEMA P-695, February. Washington, DC: FEMA.

13. Huang, Y.-N., A. S. Whittaker, and N. Luco. 2008. "Maximum Spectral Demands in the Near-Fault Region." *Earthquake Spectra* 24 (1): 319–341.

14. Beyer, K., and J. J. Bommer. 2006. "Relationships between Median Values and between Aleatory Variabilities for Different Definitions of the Horizontal Component of Motion." *Bulletin of the Seismological Society of America* 96 (4A): 1512–1522.

15. ANSI/AISC (American National Standards Institute/American Institute of Steel Construction). 2010. *Seismic Provisions for Structural Steel Buildings*. AISC 341. Chicago, IL: AISC.

16. ANSI/AISI (American Iron and Steel Institute). 2007. *Standard for Seismic Design of Cold-Formed Steel Structural Systems—Special Bolted Moment Frames*. AISI S110. Washington, DC: AISI.

17. ANSI/AISC. 2005. *Seismic Provisions for Structural Steel Buildings*. AISC 341. Chicago, IL: AISC.

18. Fischer, D., A. Filiatraut, B. Folz, C.-M. Uang, and F. Seible. 2001. *Shake Table Tests of a Two-Story Woodframe House*. CUREE-Caltech Woodframe Project publication W-06. Richmond, CA: Consortium of Universities for Research in Earthquake Engineering.

19. AF&PA (American Forest and Paper Association). 2005. *National Design Specification (NDS) for Wood Construction*. Washington, DC: AF&PA.

20. ASCE. 2002. *Minimum Design Loads for Buildings and Other Structures*. ASCE 7-02. Reston, VA: ASCE.

21. ICC. 2000. *International Building Code*. Falls Church, VA: ICC.

22. ICC. 2003. *International Building Code*. Falls Church, VA: ICC.

23. ICBO (International Conference of Building Officials). 1997. *Uniform Building Code*. Whittier, CA: ICBO.

24. FEMA. 2009. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. Washington, DC: FEMA.

25. ASCE. 2000. *Seismic Analysis of Safety-Related Nuclear Structures*. ASCE 4-98. Reston, VA: ASCE.

26. AF&PA. 2005. *Special Design Provisions for Wind and Seismic*. Washington, DC: AF&PA.

27. ANSI/AISI. 2007. *North American Standard for Cold-Formed Steel Framing—Lateral Design*. AISI S213, with supplement 1 (2009). Washington, DC: AISI.

28. FEMA. 2000. *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. FEMA 356. Washington, DC: FEMA.

29. FEMA. 2005. *Improvement of Nonlinear Static Analysis Procedures*. FEMA 440. Washington, DC: FEMA.

30. ATC (Applied Technology Council). 1996. *Seismic Evaluation and Retrofit of Concrete Buildings*. ATC 40. Redwood City, CA: ATC.

31. Ghosh, S. K., S. Dowty, and P. Dasgupta. 2010. *Significant Changes to the Seismic Load Provisions of ASCE 7-10: An Illustrated Guide*. Reston, VA: ASCE.

Notation

A_x = torsional amplification factor (ASCE 7-10 section 12.8.4.3)

C_a = seismic coefficient as set forth in Table 16-Q of the 1997 UBC[23]

C_d = deflection amplification factor as given in ASCE 7-10 Tables 12.2-1, 15.4-1, and 15.4-2

C_S = seismic response coefficient determined in ASCE 7-10 section 12.8.1.1 and 19.3.1 (dimensionless)

C_t = building period coefficient in ASCE 7-10 section 12.8.2.1

C_u = coefficient for upper limit on calculated period

F_a = short-period site coefficient (at 0.2-second period)

F_p = seismic force acting on a component of a structure as determined in ASCE 7-10 sections 12.11.1 and 13.3.1

F_{PGA} = site coefficient

F_{px} = diaphragm design force at floor level x

$F_{px,max}$ = value that F_{px} need not exceed

$F_{px,min}$ = minimum value of F_{px} that can be used in design

F_x = portion of the seismic base shear V induced at level x, as determined in ASCE 7-10 section 12.8.3

g = acceleration due to gravity

h = height of the roof above the base

h_{wall} = height of shear wall

h_{wp} = height of wall pier

I = importance factor as prescribed in ASCE 7-05 section 11.5.1

I_e = importance factor as prescribed in ASCE 7-10 section 11.5.1

k_a = multiplier for diaphragm flexibility

L_f = span of a flexible diaphragm that provides the lateral support for the wall; the span is measured between vertical elements that provide lateral support to the diaphragm in the direction considered; use zero for rigid diaphragms

L_{wall} = length of shear wall

L_{wp} = length of wall pier

P = vertical design load

Q_E = effect of horizontal seismic (earthquake-induced) forces

R = response modification coefficient as given in ASCE 7-10 Tables 12.2-1, 12.14-1, 15.4-1, or 15.4-2

S_1 = mapped MCE_R, 5%-damped, spectral response acceleration parameter at a period of 1 second as defined in ASCE 7-10 section 11.4.1

S_{DS} = design, 5%-damped, spectral response acceleration parameter at short periods as defined in ASCE 7-10 section 11.4.4

S_S = mapped MCE_R, 5%-damped, spectral response acceleration parameter at short periods as defined in ASCE 7-10 section 11.4.1

SD_1 = design, 5%-damped, spectral response acceleration parameter at a period of 1 second as defined in ASCE 7-10 section 11.4.4

T = the fundamental period of the building

T_L = long-period transition period as defined in ASCE 7-10 section 11.4.5

T_s = period at which the design spectrum transitions from its plateau to its descending branch, which varies with $1/T = S_{D1}/S_{DS}$

V = total design lateral force or shear at the base

V_t = design value of the seismic base shear as determined in ASCE 7-10 section 12.9.4

V_x = seismic design shear in story x as determined in ASCE 7-10 section 12.8.4 or 12.9.4

w_r = portion of W that is located at or assigned to roof level

w_x = portion of W that is located at or assigned to level x

W = effective seismic weight of the building as defined in ASCE 7-10 section 12.7.2.

W_p = weight of wall tributary to anchor

x = level under consideration; 1 designates the first level above the base

z = height of the anchor above the base of the structure

δ_M = maximum inelastic response displacement considering torsion, ASCE 7-10 section 12.12.3

δ_{M1} = δ_M at roof level of shorter building

δ_{M2} = δ_M at the same height of taller neighboring building

δ_{max} = maximum displacement at level x, considering torsion, ASCE 7-10 section 12.8.4.3

δ_{xe} = deflection of level x at the center of the mass at and above level x determined by an elastic analysis, ASCE 7-10 section 12.8-6

Δ = design story drift as determined in ASCE 7-10 secation 12.8.6

θ = stability coefficient for P-Δ effects as determined in ASCE 7-10 section 12.8.7

ρ = redundancy coefficient

Ω_0 = overstrength factor as defined in ASCE 7-10 Tables 12.2-1, 15.4-1, and 15.4-2

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Abstract

Major changes have taken place in the wind design, the seismic design, and the other provisions of ASCE 7-10 (referenced by the 2012 IBC) from ASCE 7-05. The changes in the seismic design provisions are presented and discussed in this paper. Chapter 14 changes are excluded from the scope of this article because chapter 14, "Material Specific Seismic Design and Detailing Requirements," is not adopted by the IBC. Changes from chapters 13, "Seismic Design Requirements for Nonstructural Components," and 15, "Seismic Design Requirements for Nonbuilding Structures," are also excluded, to avoid excessive length.

Keywords

ASCE, earthquake, ground motion, liquefaction, peak ground acceleration, response spectrum, seismic.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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