2010 Edition of ASCE 7

Minimum Design Loads for Building and Other Structures

Supplement No.1

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Use in conjunction with ASCE 7-10 Second Printing or with ASCE 7-10 First Printing and Errata 1 and 2.
(http://www.asce.org/sei/errata/)

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Chapter 12
SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

12.2.5.5 Special Moment Frames in Structures Assigned to Seismic Design Categories D through F.
For structures assigned to Seismic Design Categories D, E, or F, ~~a~~where a special moment frame is required by Table 12.2-1 due to the structural system limitations, the frame shall be continuous to the base.

A special moment frame that is used but not required by Table 12.2-1, ~~shall not~~is permitted to be discontinued above the base and supported by a more rigid system with a lower response modification coefficient, R.~~,~~ ~~unless~~ provided that the requirements of Sections 12.~~2~~3.3.~~1~~2 and 12.3.3.4 are met. ~~Where a special moment frame is required by Table 12.1-1, the frame shall be continuous to the foundation.~~

12.3.3.3 Elements Supporting Discontinuous Walls or Frames
~~Columns, beams, trusses, or slabs~~ Structural elements supporting discontinuous walls or frames of structures having horizontal irregularity Type 4 of Table 12.3-1 or vertical irregularity Type 4 of Table 12.3-2 shall be designed to resist the seismic load effects including overstrength factor of Section 2.4.3. The connections of such discontinuous ~~elements~~ walls or frames to the supporting members shall be adequate to transmit the forces for which the discontinuous ~~elements~~ walls or frames were required to be designed.

REVISE TABLE 12.3-2 VERTICAL STRUCTURAL IRREGULARITIES

4.	**In-Plane Discontinuity in Vertical Lateral-Force-Resisting Element Irregularity:** In-plane discontinuity in vertical lateral force-resisting elements irregularity 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~~ structural elements.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F

12.3.4.2 Redundancy Factor, ρ, for Seismic Design Categories D through F.
For structures assigned to Seismic Design Category D~~, E, or F~~, and having Extreme Torsional Irregularity as defined in Table 12.3-1, Type 1b, ρ shall equal 1.3. For other structures assigned to Seismic Design Category D, and for structures assigned to Seismic Design Categories E or F, ρ shall equal 1.3 unless one of the following two conditions is met, whereby ρ is permitted to be taken as 1.0. A reduction in the value of rho 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).

REVISE TABLE 12.3-1 AS FOLLOWS:

1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2 Section 12.3.4.2	E and F D B, C and D C and D C and D D B, C and D D

12.8.2.1 Approximate Fundamental Period

The approximate fundamental period, T_a, in *s*, for masonry or concrete shear wall structures not exceeding 120 feet in height is permitted to be determined from Eq. 12.8-9 as follows:

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n \quad (12.8\text{-}9)$$

where C_w is calculated from Eq. 12.8-10 as follows:

$$\cancel{C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]}} \quad (12.8\text{-}10)$$

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \frac{A_i}{\left[1 + 0.83\left(\frac{h_n}{D_i}\right)^2\right]}$$

where

A_B = area of the structure, in ft^2

A_i =web area of shear wall *i*, in ft^2

D_i =Length of shear wall *i*, ft

~~h_i = height of wall *i*, in ft~~

x =number of shear walls in the building effective in resisting lateral forces in the direction under consideration

Chapter 13
SEISMIC DESIGN REQUIREMENTS FOR NONSTRUCTURAL COMPONENTS

13.2.5 TESTING ALTERNATIVE FOR SEISMIC CAPACITY DETERMINATION

As an alternative to the analytical requirements……………..equal or exceed the seismic demands determined in accordance with Section 13.3.1 and 13.3.2. For the testing alternative, the maximum seismic demand determined in accordance with Equation 13.3-2 is not required to exceed $3.2 I_p W_p$.

13.5 Architectural Components

REVISE TABLE 13.5-1 TO ADD OVERSTRENGTH COEFFICIENTS AND CONVERT ALL EXISTING VALUES FROM DECIMAL TO FRACTIONAL FORM FOR CONSISTENCY WITH TABLE 12.2-1 (NOT SHOWN IN WITH STRIKE-OUT AND UNDERLINE TEXT FOR CLARITY).

TABLE 13.5-1 COEFFICIENTS FOR ARCHITECTURAL COMPONENTS

Architectural Component	a_p[a]	R_p	Ω_0[c]
Interior Nonstructural Walls and Partitions[b]			
Plain (unreinforced) masonry walls	1	1 ½	1 ½
All other walls and partitions	1	2 ½	2 ½
Cantilever Elements (Unbraced or braced to structural frame below its center of mass)			
Parapets and cantilever interior nonstructural walls	2 ½	2 ½	2 ½
Chimneys where laterally braced or supported by the structural frame	2 ½	2 ½	2 ½
Cantilever Elements (Braced to structural frame above its center of mass)			
Parapets	1	2 ½	2 ½
Chimneys	1	2 ½	2 ½
Exterior Nonstructural Walls[b]	1[b]	2 ½	2 ½
Exterior Nonstructural Wall Elements and Connections[b]			
Wall Element	1	2 ½	2 ½
Body of wall panel connections	1	2 ½	2 ½
Fasteners of the connecting system	1 ¼	1	1 ½
Veneer			
Limited deformability elements and attachments	1	2 ½	2 ½
Low deformability elements and attachments	1	1 ½	1 ½
Penthouses (except where framed by an extension of the building frame)	2 ½	3 ½	2 ½
Ceilings			
All	1	2 ½	2 ½
Cabinets			
Permanent floor-supported storage cabinets over 6 feet (1829 mm) tall, including contents	1	2 ½	2 ½
Permanent floor-supported library shelving, book stacks and bookshelves over 6 feet (1829 mm) tall, including contents	1	2 ½	2 ½
Laboratory equipment	1	2 ½	2 ½
Access Floors			
Special access floors (designed in accordance with Section 13.5.7.2)	1	2 ½	2 ½
All other	1	1 ½	1 ½
Appendages and Ornamentations	2 ½	2 ½	2 ½
Signs and Billboards	2 ½	3	2 ½
Other Rigid Components			
High deformability elements and attachments	1	3 ½	2 ½
Limited deformability elements and attachments	1	2 ½	2 ½
Low deformability materials and attachments	1	1 ½	1 ½
Other Flexible Components			
High deformability elements and attachments	2 ½	3 ½	2 ½

TABLE 13.5-1 COEFFICIENTS FOR ARCHITECTURAL COMPONENTS

Architectural Component	a_p[a]	R_p	Ω_0[c]
Limited deformability elements and attachments	2 ½	2 ½	2 ½
Low deformability materials and attachments	2 ½	1 ½	1 ½
Egress stairways not part of the building structure	1	2 ½	2 ½

[a] A lower value for a_p shall not be used unless justified by detailed dynamic analysis. The value for a_p shall not be less than 1. ~~1.00~~. The value of a_p = 1 is for rigid components and rigidly attached components. The value of a_p = ~~2.5~~ 2 ½ is for flexible components and flexibly attached components.

[b] Where flexible diaphragms provide lateral support for concrete or masonry walls and partitions, the design forces for anchorage to the diaphragm shall be as specified in Section 12.11.2.

[c] Overstrength as required for anchorage to concrete. See Section 12.4.3 for inclusion of overstrength factor in seismic load effect.

13.6 Mechanical and Electrical Components

REVISE TABLE 13.6-1 TO ADD OVERSTRENGTH COEFFICIENTS AND CONVERT ALL EXISTING VALUES FROM DECIMAL TO FRACTIONAL FORM FOR CONSISTENCY WITH TABLE 12.2-1 (NOT SHOWN IN WITH STRIKE-OUT AND UNDERLINE TEXT FOR CLARITY).

TABLE 13.6-1 SEISMIC COEFFICIENTS FOR MECHANICAL AND ELECTRICAL COMPONENTS

MECHANICAL AND ELECTRICAL COMPONENTS	a_p[a]	R_p[b]	Ω_0[c]
Air-side HVAC, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing.	2 ½	6	2 ½
Wet-side HVAC, boilers, furnaces, atmospheric tanks and bins, chillers, water heaters, heat exchangers, evaporators, air separators, manufacturing or process equipment, and other mechanical components constructed of high-deformability materials.	1	2 ½	2 ½
Engines, turbines, pumps, compressors, and pressure vessels not supported on skirts and not within the scope of Chapter 15.	1	2 ½	2 ½
Skirt-supported pressure vessels not within the scope of Chapter 15.	2 ½	2 ½	2 ½
Elevator and escalator components.	1	2 ½	2 ½
Generators, batteries, inverters, motors, transformers, and other electrical components constructed of high deformability materials.	1	2 ½	2 ½
Motor control centers, panel boards, switch gear, instrumentation cabinets, and other components constructed of sheet metal framing.	2 ½	6	2 ½
Communication equipment, computers, instrumentation, and controls.	1	2 ½	2 ½
Roof-mounted stacks, cooling and electrical towers laterally braced below their center of mass.	2 ½	3	2 ½
Roof-mounted stacks, cooling and electrical towers laterally braced above their center of mass.	1	2 ½	2 ½
Lighting fixtures.	1	1 ½	1 ½
Other mechanical or electrical components.	1	1 ½	1 ½
VIBRATION ISOLATED COMPONENTS AND SYSTEMS[b]			
Components and systems isolated using neoprene elements and neoprene isolated floors with built-in or separate elastomeric snubbing devices or resilient perimeter stops.	2 ½	2 ½	2 ½
Spring isolated components and systems and vibration isolated floors closely restrained using built-in or separate elastomeric snubbing devices or resilient perimeter stops.	2 ½	2	2 ½
Internally isolated components and systems.	2 ½	2	2 ½
Suspended vibration isolated equipment including in-line duct devices and suspended internally isolated components.	2 ½	2 ½	2 ½
DISTRIBUTION SYSTEMS			
Piping in accordance with ASME B31, including in-line components with joints made by welding or brazing.	2 ½	12	2 ½
Piping in accordance with ASME B31, including in-line components, constructed of high or limited deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings.	2 ½	6	2 ½
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing.	2 ½	9	2 ½

TABLE 13.6-1 SEISMIC COEFFICIENTS FOR MECHANICAL AND ELECTRICAL COMPONENTS

MECHANICAL AND ELECTRICAL COMPONENTS	a_p[a]	R_p[b]	Ω_0[c]
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high- or limited-deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings.	2 ½	4 ½	2 ½
Piping and tubing constructed of low-deformability materials, such as cast iron, glass, and nonductile plastics.	2 ½	3	2 ½
Ductwork, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing.	2 ½	9	2 ½
Ductwork, including in-line components, constructed of high- or limited-deformability materials with joints made by means other than welding or brazing.	2 ½	6	2 ½
Ductwork, including in-line components, constructed of low-deformability materials, such as cast iron, glass, and nonductile plastics.	2 ½	3	2 ½
Electrical conduit and cable trays	2 ½	6	2 ½
Bus ducts	1	2 ½	2 ½
Plumbing	1	2 ½	2 ½
Manufacturing or process conveyors (nonpersonnel).	2 ½	3	2 ½

[a] A lower value for a_p is permitted where justified by detailed dynamic analyses. The value for a_p shall not be less than 1~~1.0~~. The value of a_p equal to 1~~1.0~~ is for rigid components and rigidly attached components. The value of a_p equal to 2 ½ ~~2.5~~ is for flexible components and flexibly attached components.

[b] Components mounted on vibration isolators shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_p$ if the nominal clearance (air gap) between the equipment support frame and restraint is greater than 0.25 in. If the nominal clearance specified on the construction documents is not greater than 0.25 in., the design force is permitted to be taken as F_p.

[c] Overstrength as required for anchorage to concrete. See Section 12.4.3 for inclusion of overstrength factor in seismic load effect.

Chapter 14
MATERIAL SPECIFIC SEISMIC DESIGN AND DETAILING REQUIREMENTS

14.1.1 REFERENCE DOCUMENTS

The design, construction, and quality of steel members that resist seismic forces shall conform to the applicable requirements, as amended herein, of the following:

1. AISC 360
2. AISC 341
3. AISI S100
4. AISI S110
5. AISI S230
6. AISI S213
7. ASCE 19
8. ASCE 8
9. SJI-CJ
10. SJI-JG
11. SJI-K~~ 1.1~~
12.~~10.~~ SJI-LH/DLH~~ 1.1~~
~~11. SJI-JG-1.1~~
~~12. SJI-CJ-1.0~~

14.1.3.2 Seismic Requirements for Cold-Formed Steel Structures

Where a response modification coefficient, *R*, in accordance with Table 12.2-1 is used for the design of cold-formed steel structures, the structures shall be designed and detailed in accordance with the requirements of AISI S100, ASCE 8, and, for cold-formed steel-special bolted moment frames, AISI S110, as applicable.~~ as modified in Section 14.1.3.3.~~

~~***14.1.3.3 Modifications to AISI S110***~~

~~The text of AISI S110 shall be modified as indicated in Sections 14.1.3.3.1 through 14.1.3.3.5. Italics are used for text within Sections 14.1.3.3.1 through 14.1.3.3.5 to indicate requirements that differ from AISI S110.~~

~~*14.1.3.3.1 AISI S110, Section D1* Modify Section D1 to read as follows:~~

~~***D1 COLD-FORMED STEEL SPECIAL BOLTED MOMENT FRAMES (CFS-SBMF)***~~

~~Cold-formed steel- special bolted moment frame (CFS-SBMF) systems shall withstand significant inelastic deformations through friction and bearing at their bolted connections. Beams, columns, and connections shall satisfy the requirements in this section. CFS-SBMF systems shall be limited to one story structures, no greater than 35 feet in height, without column splices and satisfying the requirements in this section. *The CFS-SBMF shall engage all columns supporting the roof or floor above. The single size beam and single size column with the same bolted moment connection detail shall be used for each frame. The frame shall be supported on a level floor or foundation.*~~

~~*14.1.3.3.2 AISI S110, Section D1.1.1* Modify Section D1.1.1 to read as follows:~~

~~***D1.1.1 CONNECTION LIMITATIONS***~~

~~Beam-to-column connections in CFS-SBMF systems shall be bolted connections with snug-tight high-strength bolts. The bolt spacing and edge distance shall be in accordance with the limits of AISI S100, Section E3. *The 8-bolt configuration shown in Table D1-1 shall be used. The faying surfaces of the beam and column in the bolted moment connection region shall be free of lubricants or debris.*~~

~~*14.1.3.3.3 AISI S110, Section D1.2.1* Modify Section D1.2.1 and add new Section D1.2.1.1 to read as follows:~~

~~***D1.2.1 BEAM LIMITATIONS***~~

~~In addition to the requirements of Section D1.2.3, beams in CFS-SBMF systems shall be *ASTM A653 galvanized 55 ksi (374 MPa) yield stress* cold-formed *steel* C-section members with lips, and designed in accordance with Chapter C of AISI S100. *The*~~

~~*beams shall have a minimum design thickness of 0.105 in. (2.67 mm). The beam depth shall be not less than 12 in. (305 mm) or greater than 20 in. (508 mm).* The flat depth-to-thickness ratio of the web shall not exceed 6.18 E / F_y.~~

~~***D1.2.1.1 SINGLE-CHANNEL BEAM LIMITATIONS***~~

~~*When single-channel beams are used, torsional effects shall be accounted for in the design.*~~

~~*14.1.3.3.4 AISI S110, Section D1.2.2* Modify Section D1.2.2 to read as follows:~~

~~***D1.2.2 COLUMN LIMITATIONS***~~

~~In addition to the requirements of D1.2.3, columns in CFS-SBMF systems shall be *ASTM A500 Grade B* cold-formed *steel* hollow structural section (HSS) members *painted with a standard industrial finished surface*, and designed in accordance with Chapter C of AISI S100. *The column depth shall be not less than 8 in. (203 mm) or greater than 12 in. (305 mm).* The fl at depth-to-thickness ratio shall not exceed *1.40 E / F_y*.~~

~~*14.1.3.3.5 AISI S110, Section D1.3* Delete text in Section D1.3 to read as follows:~~

~~***D1.3 DESIGN STORY DRIFT***~~

~~Where the applicable building code does not contain design coefficients for CSF-SBMF systems, the provisions of Appendix 1 shall apply. For structures having a period less than *Ts*, as defined in the applicable building code, alternate methods of computing Δ shall be permitted, provided such alternate methods are acceptable to the authority having jurisdiction.~~

14.1.6 STEEL CABLES

~~The design strength of steel cables shall be determined by the requirements of ASCE 19 except as modified by this chapter. ASCE 19, Section 3.1.2(d), shall be modified by substituting 1.5(*T4*) where *T4* is the net tension in cable due to dead load, prestress, live load, and seismic load. A load factor of 1.1 shall be applied to the prestress force to be added to the load combination of Section 3.1.2 of ASCE 19.~~ The design strength of steel cables serving as main structural load carrying members shall be determined by the requirements of ASCE/SEI 19.

14.2.2 MODIFICATIONS TO ACI 318.

The text of ACI 318 shall be modified as indicated in Sections 14.2.2.1 through ~~14.2.2.9~~14.2.2.8. Italics are used for text within Sections 14.2.2.1 through ~~14.2.2.9~~14.2.2.8 to indicate requirements that differ from ACI 318.

14.2.2.1 Definitions.

Add the following definitions to Section 2.2.

DETAILED PLAIN CONCRETE STRUCTURAL WALL: *A wall complying with the requirements of Chapter 22.*

ORDINARY PRECAST STRUCTURAL WALL: *A precast wall complying with the requirements of Chapters 1 through 18.*

~~***WALL PIER:*** *A wall segment with a horizontal length-to-thickness ratio of at least 2.5, but not exceeding 6, whose clear height is at least two times its horizontal length.*~~

14.2.2.2 ACI 318, Section 7.10.

Modify Section 7.10 by revising Section 7.10.5.~~6~~7 to read as follows:

7.10.5.~~6~~7

NO CHANGE TO TEXT OF THE SECTION.

14.2.2.3 Scope.

Modify Section 21.1.1.3 to read as follows:

21.1.1.3 All members shall satisfy requirements of Chapters 1 to 19 and 22. Structures assigned to SDC B, C, D, E, or F also shall satisfy 21.1.1.4 through 21.1.1.8, as applicable, *except as modified by the requirements of Chapters 14 and 15 ASCE 7-10.*

14.2.2.4 Intermediate Precast Structural Walls:

Modify Section 21.4 by renumbering Sections 21.4.3 and 21.4.4 to Sections 21.4.~~4~~5 and 21.4.6, respectively and adding new Section~~s~~ 21.4.3, ~~21.4.5 and 21.4.6~~, to read as follows:

21.4.3 *Connections that are designed to yield shall be capable of maintaining 80 percent of their design strength at the deformation induced by design displacement, or shall use type 2 mechanical splices.*

21.4.4 *Elements of the connection that are not designed to yield shall develop at least 1.5 S_y.*

21.4.5 *In structures assigned to SDC D, E, or F, wall piers shall be designed in accordance with 21.9 or 21.13.*

~~***21.4.5*** *Wall piers in structures assigned to SDC D, E, or F shall comply with Section 14.2.2.4 of this standard.*~~

~~***21.4.6*** *Wall piers not designed as part of a moment frame in SDC C shall have transverse reinforcement designed to resist the shear forces determined from Section 21.3.3. Spacing of transverse reinforcement shall not exceed 8 in. Transverse reinforcement shall be extended beyond the pier clear height for at least 12 in.*~~

~~***Exceptions****: The preceding requirement need not apply in the following situations:*~~

~~1. *Wall piers that satisfy Section 21.13.*~~
~~2. *Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.*~~

~~*Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.*~~

~~***14.2.2.5 Wall Piers and Wall Segments.***~~

~~Modify Section 21.9 by adding a new Section 21.9.10 to read as follows:~~

~~***21.9.10 Wall Piers and Wall Segments.***~~

~~***21.9.10.1*** *Wall piers not designed as a part of a special moment-resisting frame shall have transverse reinforcement designed to satisfy the requirements in Section 21.9.10.2.*~~

~~**Exceptions:**~~

~~1. *Wall piers that satisfy Section 21.13.*~~
~~2. *Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers, and such segments have a total stiffness of at least six times the sum of the in-plane stiffnesses of all the wall piers.*~~

~~***21.9.10.2*** *Transverse reinforcement with seismic hooks at both ends shall be designed to resist the shear forces determined from Section 21.6.5.1. Spacing of transverse reinforcement shall not exceed 6 in. (152 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least 12 in. (304 mm).*~~

~~***21.9.10.3*** *Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns*~~

14.2.2.~~6~~5 Special Precast Structural Walls.

Modify Section 21.10.2 to read as follows:

21.10.2 Special structural walls constructed using precast concrete shall satisfy all the requirements of Section 21.9 in addition to Section 21.4 *as modified by Section 14.2.2 of ASCE 7-10*.

14.2.2.~~7~~6 Foundations.

NO CHANGE TO TEXT OF THIS SECTION

14.2.2.~~8~~7 Detailed Plain Concrete Shear Walls.

NO CHANGE TO TEXT OF THIS SECTION

~~***14.2.2.9 Strength Requirements for Anchors:***~~

~~Modify Section D.4 by adding a new exception at the end of Section D.4.2.2 to read as follows:~~

~~**EXCEPTION**: *If N_b is determined using Eq. D-7, the concrete breakout strength of Section D.4.2 shall be considered satisfied by the design procedure of Sections D.5.2 and D.6.2 without the need for testing regardless of anchor bolt diameter and tensile embedment.*~~

14.5.2 FRAMING

~~All wood columns and posts shall be framed to provide Full end bearing. Alternatively, column and post end connections shall be designed to resist the full compressive loads neglecting all end-bearing capacity. Continuity of wall top plates or provision for transfer of induced axial load forces shall be provided. Where offsets occur in the wall line, portions of the shear wall on each side of the offset shall be considered as separate shear walls unless provisions for force transfer around the offset are provided.~~

Chapter 15
SEISMIC DESIGN REQUIREMENTS FOR NONBUILDING STRUCTURES

15.4 Structural Design Requirements

REVISE TABLE 15.4-2 AS SHOWN BELOW:

Nonbuilding Structure Type	Detailing Requirements[c]	R	Ω_0	C_d	B	C	D	E	F
All other nonreinforced masonry structures not similar to buildings	14.4.1	1.25	2	1.5	NL	~~NL~~NP	NP	NP	NP

15.5.3 STEEL STORAGE RACKS

Steel storage racks supported at or below grade shall be designed in accordance with ANSI/RMI MH 16.1 and its force and displacement requirements, except as follows:

15.5.3.1

~~Modify Section 2.6.2 of ANSI/RMI MH 16.1 as follows:~~

~~2.6.2 MINIMUM SEISMIC FORCES~~

~~The storage rack shall be designed…~~

*~~**Above-Grade Elevation:** Storage rack installed at elevations above grade shall be designed, fabricated, and installed in accordance with the following requirements:~~*

~~Storage racks shall meet the force and displacement requirements required of nonbuilding structures supported by other structures, including the force and displacement effects caused by amplifications of upper-story motions. In no case shall the value of V be taken as less than the value of Fp determined in accordance with Section 13.3.1 of ASCE/SEI 7, where Rp is taken equal to R, and ap is taken equal to 2.5.~~

15.5.3.~~2~~1

Modify Section ~~7.2.2~~ 7.1.2 of ANSI/RMI MH 16.1 as follows:

~~7.2.2~~ 7.1.2 Base Plate Design

NO CHANGE TO TEXT OF THIS SECTION

15.5.3.~~3~~2

Modify Section ~~7.2.4~~ 7.1.4 of ANSI/RMI MH 16.1 as follows:

NOTE: REPLACE ENTIRE TEXT OF 7.1.4. STRIKEOUT AND UNDERLINE NOT SHOWN WITH EXISTING TEXT BECAUSE MODIFIED PROVISION USES STRIKEOUT AND UNDERLINE.

~~7.2.4~~ 7.1.4 Shims

Shims may be used under the base plate to maintain the plumbness and/or levelness of the storage rack. The shims shall be made of a material that meets or exceeds the design bearing strength (LRFD) or allowable bearing strength (ASD) of the floor. The shim size and location under the base plate shall be equal to or greater than the required base plate size and location.

~~In no case shall the total thickness of a shim stack under a base plate exceed six times the diameter of the largest anchor bolt used in that base.~~

Shims stacks ~~having a total thickness greater than two and less than or equal to six times the anchor bolt diameter under bases with only one anchor bolt~~ shall be interlocked or welded together in a fashion that is capable of transferring all the shear forces at the base.

~~Shims stacks having a total thickness of less than or equal to two times the anchor bolt diameter need not be interlocked or welded together.~~

Bending in the anchor associated with shims or grout under the base plate shall be taken into account~~, if necessary,~~ in the design of anchor bolts.

15.7.6 GROUND-SUPPORTED STORAGE TANKS FOR LIQUIDS

REVISE EQUATION 15.7-10

For $T_c \le T_L$:

$$S_{ac} = \frac{1.5S_{D1}}{T_c} \leq \cancel{1.5}S_{DS} \qquad (15.7\text{-}10)$$

Chapter 16
SEISMIC RESPONSE HISTORY PROCEDURES

16.1.4 RESPONSE PARAMETERS

For each ground motion analyzed, the individual response parameters shall be multiplied by the following scalar quantities:

a. Force response parameters shall be multiplied by I_e/R, where I_e is the importance factor determined in accordance with Section 11.5.1 and R is the Response Modification Coefficient selected in accordance with Section 12.2.1.
b. Drift quantities shall be multiplied by C_d/R, where C_d is the deflection amplification factor specified in Table 12.2-1.

For each ground motion i, where i is the designation assigned to each ground motion, the maximum value of the base shear, V_i, member forces, Q_{Ei}, and story drifts, Δ_i, at each story scaled as indicated in the preceding text ~~and story drifts, Δ_i, at each story as defined in Section 12.8.6~~ shall be determined. Story drifts at each story shall be determined at the locations defined in Section 12.8.6. ~~Where the maximum scaled base shear predicted by the analysis, V_i, is less than 85 percent of the value of V determined using the minimum value of C_s set forth in Eq. 12.8-5 or when located where S_1 is equal to or greater than 0.6g, the minimum value of C_s set forth in Eq. 12.8-6, the scaled member forces, Q_{Ei}, shall be additionally multiplied by V / V_i where V is the minimum base shear that has been determined using the minimum value of C_s set forth in Eq. 12.8-5, or when located where S_1 is equal to or greater than 0.6g, the minimum value of C_s set forth in Eq. 12.8-6. Where the maximum scaled base shear predicted by the analysis, V_i, is less than 0.85C_sW, where C_s is from Eq. 12.8-6, drifts shall be multiplied by 0.85 $C_s W / V_i$.~~

16.1.4.1 Additional Scaling of Forces

Where the maximum scaled base shear predicted by the analysis, V_i, is less than 85 percent of the calculated base shear, V, using the equivalent lateral force procedure, the scaled member forces, Q_{Ei}, shall be additionally multiplied by 0.85V/V_i.

Where V = the equivalent lateral force procedure base shear, calculated in accordance with Section 12.8.

16.1.4.2 Additional Scaling of Drifts

Where the maximum scaled base shear predicted by the analysis, V_i, is less than 0.85C_sW, where C_s is determined in accordance with Section 12.8.1.1, the scaled story drifts, Δ_i, shall be additionally multiplied by 0.85C_sW/V_i.

Chapter 21
SITE-SPECIFIC GROUND MOTION PROCEDURES FOR SEISMIC DESIGN

21.2 RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION HAZARD ANALYSIS

The requirements of Section 21.2 shall be satisfied where a ground motion hazard analysis is performed or required by Section 11.4.7. The ground motion hazard analysis shall account for the regional tectonic setting, geology, and seismicity, the expected recurrence rates and maximum magnitudes of earthquakes on known faults and source zones, the characteristics of ground motion attenuation, near source effects, if any, on ground motions, and the effects of subsurface site conditions on ground motions. The characteristics of subsurface site conditions shall be considered either using attenuation relations that represent regional and local geology or in accordance with Section 21.1. The analysis shall incorporate current seismic interpretations, including uncertainties for models and parameter values for seismic sources and ground motions. If the spectral response accelerations predicted by the attenuation relations do not represent the maximum response in the horizontal plane, then the response spectral accelerations computed from the hazard analysis shall be scaled by factors to increase the motions to the maximum response. If the attenuation relations predict the geometric mean or similar metric of the two horizontal components, then the scale factors shall be: 1.1 for periods less than or equal to 0.2 sec; 1.3 for a period of 1.0 sec., and, 1.5 for periods greater than or equal to 5.0 sec., unless it can be shown that other scale factors more closely represent the maximum response, in the horizontal plane, to the geometric mean of the horizontal components. Scale factors between these periods shall be obtained by linear interpolation. The analysis shall be documented in a report.

Chapter 23
SEISMIC DESIGN REFERENCE DOCUMENTS

ACI 318
Sections 13.4.2.1, 13.4.4, 13.5.7.2, 14.2.2, 14.2.2.1, 14.2.2.2, 14.2.2.3, 14.2.2.4, 14.2.2.5, 14.2.2.6, 14.2.2.7, ~~14.2.2.8,~~ ~~14.2.2.9~~, 14.2.3, 14.2.3.1.1, 14.2.3.2.1, 14.2.3.2.2, 14.2.3.2.3, 14.2.3.2.5, 14.2.3.2.6, 14.3.1, 14.4.4.2.2, 14.4.5.2, 15.4.9.1, 15.6.2, 15.7.5, 15.7.11.7.
Building Code Requirements for Structural Concrete and Commentary ~~(2008)~~(2011)

ACI 530/ASCE 5/TMS 402
Sections 13.4.2.2, 14.4.1, 14.4.2, 14.4.3, 14.4.3.1, 14.4.4.1, 14.4.4.2.2, 14.4.5, 14.4.5.1, 14.4.5.3, 14.4.5.4, 14.4.5.5, 14.4.5.6, 14.4.6, 14.4.6.1, 14.4.6.2.2, 14.4.7, 14.4.7.1, 14.4.7.2, 14.4.7.3, 14.4.7.4, 14.4.7.5, 14.4.7.6, 14.4.8, 14.4.8.1, 15.4.9.2
Building Code Requirements for Masonry Structures,~~2008~~2011

ACI 530.1/ASCE 6/TMS 602
Sections 14.4.1, 14.4.2, 14.4.7, 14.4.7.1
Specification for Masonry Structures, ~~2008~~2011

~~**AF&PA**~~
~~**American Forest and Paper Association**~~
~~**1111 19th Street NW, Suite 800**~~
~~**Washington, DC 20036**~~

AWC
American Wood Council
803 Sycolin Road, Suite 201
Leesburg, VA 20175

~~**AF&PA**~~ **AWC NDS**
Section 12.4.3.3, ~~12.14.2.2.2.3,~~ 12.14.3.2.3, 14.5.1
National Design Specification for Wood Construction, Including Supplements,
~~AF&PA NDS-05, 2005~~ AWC NDS-12, 2012

~~**AF&PA**~~ **AWC SDPWS**
Section ~~12.14.6.2,~~ 12.14.7.2, 14.5.1~~, 14.5.3, 14.5.3.1~~
~~AF&PA~~ Special Design Provisions for Wind and Seismic, AWC SDPWS-08, 2008 (previously AF&PA SDPWS-08)

ANSI/AISI S100 W/S2-10
Sections 14.1.1, 14.1.3.1, 14.1.3.2, 14.1.3.3.2, 14.1.3.3.3, 14.1.3.3.4, 14.1.4.1, 14.1.5
North American Specification for the Design of Cold-Formed Steel Structural Members, 2007, with Supplement 2, 2010

ANSI/AISI S110 W/S1-09
Sections 14.1.1, 14.1.3.2, ~~14.1.3.3, 14.1.3.3.1, 14.1.3.3.2, 14.1.3.3.3, 14.1.3.3.4, 14.1.3.3.5,~~ Table 12.2-1
Standard for Seismic Design of Cold-Formed Steel Structural Systems—Special Bolted Moment Frames, 2007, with Supplement 1, 2009

API 12B
Section 15.7.8.2
Specification for Bolted Tanks for Storage of Production Liquids, ~~Specification~~ 12B ~~14th~~15th edition, ~~1995~~2009

API 620
Sections 15.4.1, 15.7.8.1, 15.7.13.1
Design and Construction of Large, Welded, Low Pressure Storage Tanks, 11th edition, Addendum ~~1~~2, ~~2009~~2010

API 650
Sections 15.4.1, 15.7.8.1, 15.7.9.4
Welded Steel Tanks for Oil Storage, 11th e~~E~~dition,
Addendum ~~1~~3, ~~2008~~2011

API 653
Section 15.7.6.1.9
Tank Inspection, Repair, Alteration, and Reconstruction, ~~3rd~~ 4th edition, Addendum 1, ~~2001~~2010

ASCE 5
~~Sections 13.4.2.2, 14.4.1, 14.4.2, 14.4.3, 14.4.3.1, 14.4.4.1, 14.4.4.2.2, 14.4.5, 14.4.5.1, 14.4.5.3, 14.4.5.4, 14.4.5.5, 14.4.5.6, 14.4.6, 14.4.6.1, 14.4.6.2.2, 14.4.7, 14.4.7.1, 14.4.7.2, 14.4.7.3, 14.4.7.4, 14.4.7.5, 14.4.7.6, 14.4.8, 14.4.8.1, 15.4.9.2~~
~~*Building Code Requirements for Masonry Structures,*2008~~

ASCE 6
~~Sections 14.4.1, 14.4.2, 14.4.7, 14.4.7.1~~
~~*Specification for Masonry Structures,* 2008~~

ASCE/SEI 19
Sections 14.1.1, 14.1.6
Structural Applications for Steel Cables for Buildings, ~~1996~~2010

ASME A17.1
Sections 13.6.10, 13.6.10.3
Safety Code for Elevators and Escalators, ~~*2004*~~ *2007*

ASME B31 (CONSISTS OF THE FOLLOWING LISTED STANDARDS)
Sections 13.6.5.1, 13.6.8.1, 16.8.4, Table 13.6-1
Power Piping, ASME B31.1, ~~2001~~ 2010
Process Piping, ASME B31.3, ~~2002~~ 2010
~~*Liquid*~~ *Pipeline Transportation Systems for Liquid Hydrocarbons and Other Liquids,* ~~*Liquid Petroleum Gas, Anhydrous Ammonia, and Alcohols,*~~ ASME B31.4, ~~2002~~ 2009
Refrigeration Piping and Heat Transfer Components, ASME B31.5, ~~2001~~ 2010
Gas Transmission and Distribution Piping Systems, ASME B31.8, ~~1999~~ 2010
Building Services Piping, ASME B31.9, ~~1996~~ 2008
Slurry Transportation Piping Systems, ASME B31.11, 2002
Hydrogen Piping and Pipelines, ASME B31.12, 2008
Standard for the Seismic Design and Retrofit of Above-Ground Piping Systems, ASME B31Ea-2010

ASME BPVC-~~01~~ (CONSISTS OF THE FOLLOWING LISTED STANDARDS)
Sections 13.6.9, 13.6.11, 15.7.11.2, 15.7.11.6, 15.7.12.2
~~*Boiler and Pressure Vessel Code*, 2004 excluding Section III, Nuclear Components, and Section XI, In-Service Inspection of Nuclear Components~~
Rules for Construction of Power Boilers, BPVC-I 2010
Rules for Construction of Heating Boilers, BPVC-IV 2010
Rules for Construction of Pressure Vessels, BPVC-VIII Division 1 2010
Rules for Construction of Pressure Vessels, BPVC-VIII Division 2 Alternative Rules 2010
Rules for Construction of Pressure Vessels, BPVC-VIII Division 3 Alternative Rules for Construction of High Pressure Vessels 2010

AWWA D103
Sections 15.4.1, 15.7.7.2, 15.7.9.5
Factory-Coated Bolted Steel Tanks for Water Storage,
~~1997~~2009

NFPA 59A
Section 15.4.8
Standard for the Production, Storage, and Handling of Liquefied natural Gas (LNG),
~~2006~~2009

ICC
~~**International Code Council**~~
~~**5203 Leesburg Pike**~~

~~**Suite 600**~~
~~**Falls Church, VA 22041**~~
500 New Jersey Ave. NW
6th Floor
Washington, DC 20001

*** IRC**
Section 11.1.2
2012~~2003~~ International Residential Code, 2012~~2003~~

RMI
ANSI/MH 16.1
Section 15.5.3
Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks, ~~2008~~ 2011

SJI

ANSI/SJI-CJ-2010
Section 14.1.1
Standard Specification for Composite Steel Joists, CJ-series, 2010

ANSI/SJI-JG-2010
Section 14.1.1
Standard Specification for Joist Girders, 2010

ANSI/SJI-K~~-1.1~~-2010
Section 14.1.1
Standard Specification~~s~~ for Open Web Steel Joists,
K-Series, ~~2005~~2010

ANSI/SJI-LH/DLH~~-1.1~~-2010
Section 14.1.1
Standard Specification~~s~~ for Longspan Steel Joists, LH-Series and Deep Longspan Steel Joists, DLH-Series, 2010~~2005~~

ANSI/SJI-JG-1.1
~~Section 14.1.1~~
~~*Standard Specifications for Joist Girders*, 2005~~

ANSI/SJI-CJ-1.0
~~Section 14.1.1~~
~~*Standard Specifications for Composite Steel Joists,*~~
~~2006~~

TMS 402
~~Sections 13.4.2.2, 14.4.1, 14.4.2, 14.4.3, 14.4.3.1,~~
~~14.4.4.1, 14.4.4.2.2, 14.4.5, 14.4.5.1, 14.4.5.3, 14.4.5.4, 14.4.5.5, 14.4.5.6, 14.4.6, 14.4.6.1, 14.4.6.2.2, 14.4.7, 14.4.7.1, 14.4.7.2, 14.4.7.3, 14.4.7.4, 14.4.7.5, 14.4.7.6, 14.4.8, 14.4.8.1, 15.4.9.2~~
~~*Building Code Requirements for Masonry Structures*,2008~~

TMS 602
~~Sections 14.4.1, 14.4.2, 14.4.7, 14.4.7.1~~
~~*Specification for Masonry Structures*, 2008~~

COMMENTARY

Chapter C7
SNOW LOADS

C7.2 GROUND SNOW LOADS, p_g

ADD NEW PARAGRAPH ABOVE 3RD TO LAST PARAGRAPH

Regardless of the methodology used to obtain ground snow loads (e.g., Figure 7.3-1, a case study or a State study), the ASCE 7 snow load provisions should be used to obtain the ground-to-roof conversion, unbalanced loads, drift loads, and related items.

C7.4 SLOPED ROOF SNOW LOADS, p_s

INSERT NEW LAST PARAGRAPH

If the roof has snow retention devices (installed to prevent snow and ice from sliding off the roof), it should be considered an obstructed roof and the slope factor Cs should be based on the “All Other Surfaces” curves in Figure 7-2.

C7.6 UNBALANCED ROOF SNOW LOADS

C7.6.1 Unbalanced Snow Loads for Hip and Gable Roofs.

ADD THE FOLLOWING TWO PARAGRAPHS AT THE END OF THE SECTION

For intersecting gable roofs and similar roof geometries, some codes and standards have required a valley drift load. Such valley drift loads are not required in ASCE 7. However valley locations are subject to unbalanced or gable roof drifts as described in Section 7.6.1. An example of unbalanced loading on an L-shaped gable roof is presented in O’Rourke (2007).

For intersecting monoslope roofs and intersecting gable roofs with slopes greater than 7 on 12, unbalanced loads are not required in ASCE 7. However at such valleys, snow on each side of the valley is prevented from sliding by the presence of roof snow on the other side of the valley. As such, the valley portion of the roof (drainage area up-slope of the re-entrant corner) is obstructed and the Slope Factor Cs should be based on the “All Other Surfaces” lines in Figure 7-2.

C7.7 DRIFTS ON LOWER ROOFS (AERODYNAMIC SHADE)

ELIMINATE THE LAST SENTENCE IN THE 8 TH PARAGRAPH OF THE SECTION. ADD THE FOLLOWING NEW PARAGRAPH IMMEDIATELY AFTER THE CURRENT 8 TH PARAGRAPH:

The drift load provisions cover most, but not all, situations. Finney (1939) and O’Rourke (1989) document a larger drift than would have been expected based on the length of the upper roof. The larger drift was caused when snow on a somewhat lower roof, upwind of the upper roof, formed a drift between those two roofs allowing snow from the upwind lower roof to be carried up onto the upper roof then into the drift on its downwind side. It was suggested that the sum of the lengths of both roofs could be used to calculate the size of the leeward drift. ~~The issue of potential reduction in leeward drift size at a roof step due to a parapet wall is discussed in O’Rourke (2007).~~

Generally, the addition of a parapet wall on a high roof cannot be relied upon to substantially reduce the leeward snow drift loading on an adjacent or adjoining lower roof. This is particularly true for the case of a single parapet wall of typical height located at the roof step. Also, the addition of a parapet wall at a roof step would increase the space available for windward drift formation on the lower roof. The issue of potential reduction in leeward drift size at a roof step due to a parapet wall is discussed in more detail in O’Rourke (2007).

C7.8 ROOF PROJECTIONS AND PARAPETS

NEW PARAGRAPH AT THE END OF THE SECTION.

Refer to Section C7.7 for more description of the effects that a parapet wall at a high roof can have on the snow drift loading at an adjacent or adjoining lower roof.

C7.9 SLIDING SNOW

IN THE LAST PARAGRAPH REPLACE "SNOW GUARDS" WITH "SNOW RETENTION DEVICES" IN 4 PLACES.

~~Snow guards~~ Snow retention devices are needed on some roofs to prevent roof damage and eliminate hazards associated with sliding snow (Tobiasson et al. 1996). When ~~snow guards~~ snow retention devices are added to a sloping roof, snow loads on the roof can be expected to increase. Thus, it may be necessary to strengthen a roof before adding ~~snow guards~~ snow retention devices. When designing a roof that will likely need ~~snow guards~~ snow retention devices in the future, it may be appropriate to use the "all other surfaces" curves in Fig. 7-2 not the "unobstructed slippery surfaces" curves.

C7.13 OTHER ROOFS AND SITES

REPLACE EXISTING EXAMPLE 1 WITH THE FOLLOWING; STRIKEOUT AND UNDERLINE TEXT OMITTED FOR CLARITY:

Example 1: Determine balanced and unbalanced design snow loads for an apartment complex in a suburb of Hartford, Connecticut. Each unit has a 6-on-12 slope unventilated gable roof. The building length is 100 ft (30.5 m) and the eave to ridge distance, W, is 30 ft (9.1 m). Composition shingles clad the roofs. Trees will be planted among the buildings.

Flat-Roof Snow Load:

$$p_f = 0.7C_eC_tI_sp_g$$

where

p_g = 30 lb/ft^2 (1.44 kN/m^2) (from Fig. 7-1)

C_e = 1.0 (from Table 7-2 for Terrain Category B and a partially exposed roof)

C_t = 1.0 (from Table 7-3); and I_s = 1.0 (from Section 7.3.3 and Table 1.5-2).

Thus:

p_f = (0.7)(1.0)(1.0)(1.0)(30) = 21 lb/ft^2 (balanced load)

in SI: $p_f = (0.7)(1.0)(1.0)(1.0)(1.44) = 1.01$ kN/m^2

Because the roof slope is greater than 15°, the minimum roof snow load, p_m , does not apply (see Section 7.3.4).

Sloped-Roof Snow Load:

p_s = $C_s\, p_f$ where $C_s = 1.0$ (using the "All Other Surfaces" (or solid) line, Fig. 7-2a).

Thus:

p_s = 1.0(21) = 21 lb/ft^2

(in SI: $p_s = 1.0(1.01) = 1.01$ kN/m^2)

Unbalanced Snow Load: Because the roof slope is greater than ½ on 12 (2.38°), unbalanced loads must be considered. For p_g = 30 psf (1.44 kN/m^2) and $W = \ell_u$ = 30 ft (9.14 m), h_d = 1.86 ft (0.57 m) from Fig. 7-9 and γ = 17.9 pcf (2.80 kN/m^3) from Eq. 7.7-1. For a 6 on 12 roof, S = 2.0 and hence the intensity of the drift surcharge, $h_d\gamma/\sqrt{S}$, is 23.5 psf (1.13 kN/m^2) and its horizontal extent $8\sqrt{S}\, h_d/3$ is 7.0 ft (2.14 m). On the windward side, $0.3p_s$ = 6.3 psf (0.3 kN/m^2)

Rain on Snow Surcharge: A rain-on-snow surcharge load need not be considered because p_g > 20 psf (0.96 kN/m^2) (see Section 7.10). See Fig. C7-5 for both loading conditions.