2005 Edition of ASCE 7
Minimum Design Loads for Buildings and Other Structures
Includes Supplement No. 1

Errata

This document containing the errata to ASCE 7-05 is periodically updated and posted on the SEI website at www.SEInstitute.org The errata are organized by date in descending order (most recent to furthest past) hence regular users of this document need only review the errata posted since their previous use. This document will be continuously updated as additional errata are identified.

Most recent errata posting: May 3, 2007
Third errata posting: October 3, 2006
Second errata posting: September 15, 2006
First errata posting: January 6, 2006

Errata Posted on May 3, 2007

Chapter 6, Section 6.2, page 22:

Revise definition of Mean Roof Height as shown below:

MEAN ROOF HEIGHT, *h*: The average of the roof eave height and the height to the highest point on the roof surface, except that, for roof angles of less than or equal to 10°, the mean roof height shall be the roof eave ~~heave~~ height.

Chapter 6, Equation 6-13, page 27:

Revise the equation for R_ℓ as follows:

$R_\ell = R_B$ setting $\eta = 4.6 n_1 \cancel{E}\, B / \bar{V}_{\bar{z}}$

(remove the symbol "E" – it was included by mistake)

Chapter 6, Figure 6-18D, page 69 – two changes:

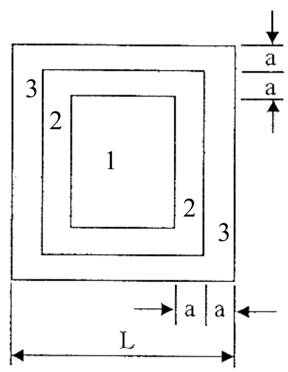
Change the title of Figure 6-18D from "Troughed Free Roofs" to "Free Roofs" as it applies to monosloped, pitched and troughed free roofs. Also, in the title block change " **λ = 0°, 180°** " to read " **γ = 90°** ."

Revise Note 6 as shown below:

6. Notation:
   - L : horizontal dimension of roof, measured in the along wind direction, ft. (m)
   - h : mean roof height, ft. (m). See Figures 6-18A, B or C for a graphical depiction of this dimension.
   - γ : direction of wind, degrees
   - θ : angle of plane of roof from horizontal, degrees

Chapter 6, Figure 6-19A, page 70:

Revise the illustration in the upper left-hand corner to correctly identify the location of the zones. Correct illustration is provided below:



Chapter 6, Figure 6-19B, page 71:

Change the title of Figure 6-19B from "Monoslope Free Roofs" to "Pitched Free Roofs."

Chapter 6, Figures 6-19B and 6-19C, pages 71 and 72:

Dimension "a" for Figures 6-19B and 6-19C is as shown in Figure 6-19A. Please also see Note 6 below.

Chapter 6, Figure 6-21, page 74:

The value of C_f for h/D=25 and Round ($D\sqrt{q_z}$ >2.5), Very rough (D'/D = 0.08) is 1.2, not 0.2 as shown.

Chapter 6, Figure 6-23, page 76:

Revise Note 6 as follows:

6. Loads due to ice accretion as described in Chapter 10 ~~Section 11~~ shall be accounted for.

Chapter 7, page 81:

Revise Section 7.1 as by removing the definition for "*L*":

7.1 SYMBOLS AND NOTATION

~~*L* = roof length parallel to the ridge line, in ft (m)~~

Chapter 7, Figure 7-2b, page 86:

The dashed line for "Unobstructed Slippery Surfaces", $C_t = 1.1$ should begin at 10° degrees, not at 15° as presently drawn.

Chapter 7, Figure 7-4, page 88:

Add a support symbol ▲, between second and third support symbol in Case 1, similar as shown in Case 2 and Case 3.

Chapter 7, Figure 7-5, page 89:

For the middle loading diagram, change the title to "Unbalanced W ≤ 20 ft with roof rafter system"

(The "less than" symbol should be a "less than or equal to" symbol")

Chapter 7, Table 7.2, page 92:

Revise Table 7.2 heading as shown below – "Exposure of roof[a]" applies to all three columns:

Terrain Category	Exposure of Roof[a]		
	Fully Exposed	~~**Exposure of Roof[a]**~~ **Partially Exposed**	**Sheltered**

Chapter 12, Table 12.2-1, pages 120-122:

Revise Table 12.2-1 as shown below:

TABLE 12.2-1 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE–RESISTING SYSTEMS

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls	14.2 ~~and 14.2.3.6~~	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls	14.2 ~~and 14.2.3.4~~	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls	14.2 ~~and 14.2.3.2~~	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls	14.2 ~~and 14.2.3.1~~	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls	14.2 ~~and 14.2.3.5~~	4	2½	4	NL	NL	40[k]	40[k]	40[k]
6. Ordinary precast shear walls	14.2 ~~and 14.2.3.3~~	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4 ~~and 14.4.3~~	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4 ~~and 14.4.3~~	3½	2½	2¼	NL	NL	NP	NP	NP
Systems 9 thru 12 omitted for brevity.									
13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1~~, 14.1.4.2,~~ and 14.5	6½	3	4	NL	NL	65	65	65
14. Light-framed walls with shear panels of all other materials	14.1~~, 14.1.4.2,~~ and 14.5	2	2½	2	NL	NL	35	NP	NP
15. Light-framed wall systems using flat strap bracing	14.1~~, 14.1.4.2,~~ and 14.5	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
Systems 1 thru 4 omitted for brevity.									
5. Special reinforced concrete shear walls	14.2 ~~and 14.2.3.6~~	6	2½	5	NL	NL	160	160	100
6. Ordinary reinforced concrete shear walls	14.2 ~~and 14.2.3.4~~	5	2½	4½	NL	NL	NP	NP	NP
7. Detailed plain concrete shear walls	14.2 ~~and 14.2.3.2~~	2	2½	2	NL	NP	NP	NP	NP
8. Ordinary plain concrete shear walls	14.2 ~~and 14.2.3.1~~	1½	2½	1½	NL	NP	NP	NP	NP
9. Intermediate precast shear walls	14.2 ~~and 14.2.3.5~~	5	2½	4½	NL	NL	40[k]	40[k]	40[k]
10. Ordinary precast shear walls	14.2 ~~and 14.2.3.3~~	4	2½	4	NL	NP	NP	NP	NP
Systems 11 thru 22 omitted for brevity.									

23. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1~~, 14.1.4.2,~~ and 14.5	7	2½	4½	NL	NL	65	65	65
24. Light-framed walls with shear panels of all other materials	14.1~~, 14.1.4.2,~~ and 14.5	2½	2½	2½	NL	NL	35	NP	NP
Systems 25 thru 27 omitted for brevity.									

Only Notes with errata are shown below.

[b]~~Reflection~~ Deflection amplification factor, C_d, for use in Sections 12.8.6, 12.8.7, and 12.9.2

[g]The tabulated value of the overstrength factor, Ω_0, is permitted to be reduced by subtracting one-half for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure except cantilever column systems.

Chapter 12, page 123:

Revise Section 12.2.5.5 as shown below:

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 special moment frame that is used but not required by Table ~~12.1-1~~ 12.2-1 shall not be discontinued and supported by a more rigid system with a lower response modification coefficient, *R*, unless the requirements of Sections 12.3.3.2 and 12.3.3.4 are met. Where a special moment frame is required by Table ~~12.1-1~~ 12.2-1, the frame shall be continuous to the foundation.

Chapter 12, page 127:

Revise Section 12.4.3.2 as shown below:

12.4.3.2 Load Combinations with Overstrength Factor. Where the seismic load effect with overstrength, E_m, defined in Section 12.4.3 is combined with the effects of other loads as set forth in Chapter 2, the following seismic load combination for structures not subject to flood or atmospheric ice loads shall be ~~shall be~~ used in lieu of the seismic load combinations in either Section 2.3.2 or 2.4.1:

Chapter 12, page 135:

Revise Section 12.13.6.5 as shown below:

12.13.6.5 Pile Anchorage Requirements. In addition to the requirements of Section ~~12.3.5.3~~ 12.13.5.3, anchorage of piles shall comply with this section. Design of anchorage of piles into the pile cap shall consider the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. For piles required to resist uplift forces or provide rotational restraint, anchorage into the pile cap shall be capable of developing the following:

Chapter 12, page 133:

Revise Section 12.10.2.1 as shown below:

12.10.2.1 Collector Elements Requiring Load Combinations with Overstrength Factor for Seismic Design Categories C through F. In structures assigned to Seismic Design Category C, D, E, or F, collector elements (see Fig. 12.10-1), splices, and their connections to resisting elements shall resist the load combinations with overstrength factor of Section 12.4.3.2.

> **EXCEPTION:** In structures or portions thereof braced entirely by light-frame shear walls, collector elements, splices, and connections to resisting elements need only be designed to resist forces in accordance with Section 12.10.1.1.

Chapter 12 – Section 12.14.1.1, page 137:

Revise item 8 as follows – also see errata posting on January 6, 2007 below:

8. For buildings with a diaphragm that is not flexible, the distance between the center of rigidity and the center of mass parallel to each major axis shall not exceed 15 percent of the greatest width of the diaphragm parallel to that axis. In addition, the following two equations shall be satisfied ~~for each major axis direction~~:

$$\sum_{i=1}^{m} k_{1i}d_{1i}^{2} + \sum_{j=1}^{n} k_{2j}d_{2j}^{2} \geq 2.5\left(0.05+\frac{e_1}{b_1}\right)b_1^{2}\sum_{i=1}^{m} k_{1i} \qquad \text{(Eq. 12.14-2A)}$$

$$\sum_{i=1}^{m} k_{1i}d_{1i}^{2} + \sum_{j=1}^{n} k_{2j}d_{2j}^{2} \geq 2.5\left(0.05+\frac{e_2}{b_2}\right)b_2^{2}\sum_{j=1}^{m} k_{1j} \qquad \text{(Eq. 12.14-2B)}$$

(Note: the subscripts on "d" in both equations and for the third summation in 12.14-2B were incorrect in the January 6th errata posting.)

Chapter 13, Sections 13.1.1 and 13.1.5, page 143:

Revise sections as follows:

13.1.1 Scope. This chapter establishes minimum design criteria for nonstructural components that are permanently attached to structures and for their supports and attachments. Where the weight of a nonstructural component is greater than or equal to 25 percent of the effective seismic weight, *W*, **of the structure as** defined in Section 12.7.2, the component shall be classified as a nonbuilding structure and shall be designed in accordance with Section 15.3.2.

13.1.5 Application ~~Applicability~~ of Nonstructural Component Requirements to Nonbuilding Structures. ~~Where the weight of a nonstructural component is greater than or equal to 25 percent of the effective seismic weight, *W*, defined in Section 12.7.2, the component shall be classified as a nonbuilding structure and shall be designed in accordance with Section 15.3.2.~~
Nonbuilding structures (including storage racks and tanks) that are supported by other

structures shall be designed in accordance with Chapter 15. Where Section 15.3 requires that seismic forces be determined in accordance with Chapter 13 and values for R_p are not provided in Table 13.5-1 or 13.6-1, R_p shall be taken as equal to the value of R listed in Section 15. The value of a_p shall be determined in accordance with footnote *a* of Table 13.5-1 or 13.6-1.

Chapter 13, Section 13.1.4, page 143:

Revise Exception 5 as shown below – also see errata posted on September 15, 2006 below:

5. Mechanical and electrical components in Seismic Design Categories D, E, ~~and~~ or F where the component importance factor, I_p, is equal to 1.0 and both of the following conditions apply:

   a. Flexible connections between the components and associated ductwork, piping, and conduit are provided, and
   b. The components weigh 20 lb (89 N) or less or, for distribution systems, weighing 5 lb/ft (73 N/m) or less.

Chapter 13, Section 13.5.6.2.2, page 147:

Revise items d and e as follows:

d. For ceiling areas exceeding 2,500 ft^2 (232 m^2), a seismic separation joint or full height partition that breaks the ceiling up into areas not exceeding 2,500 ft^2 shall be provided unless structural analyses are performed of the ceiling bracing system for the prescribed seismic forces that demonstrate ceiling system penetrations and closure angles provide sufficient clearance to accommodate the anticipated lateral displacement. Each area shall be provided with closure angles in accordance with ~~item 2~~ item b and horizontal restraints or bracing in accordance with ~~item 3~~ item c.

e. Except where rigid braces are used to limit lateral deflections, sprinkler heads and other similar type penetrations that do not behave integrally with the ceiling system in the lateral direction shall have a 2 in. (50 mm) oversize ring, sleeve, or adapter through the ceiling tile to allow for free movement of at least 1 in. (25 mm) in all horizontal directions. Alternatively, a swing joint that can accommodate 1 in. (25 mm) of ceiling movement in all horizontal directions is permitted to be provided at the top of the sprinkler head extension.

Chapter 14, Section 14.1.4.1, page 153:

Revise Section 14.1.4.1 as shown below:

14.1.4.1 Light-Framed Cold-Formed Steel Construction. Lightframed cold-formed steel construction shall be designed in accordance with AISI NAS ~~and AISI PM~~, AISI GP, AISI WSD, AISI Lateral, or ASCE 8.

Chapter 14, page 158:

Revise Section 14.3.1 as shown below:

14.3.1 Reference Documents. The design, construction, and quality of composite steel and concrete components that resist seismic forces shall conform to the applicable requirements of

1. ACI 318 excluding Chapter 22
2. ~~AISC LRFD~~ AISC 360
3. ~~AISC Seismic~~ AISC 341

Chapter 15, Table 15.4-2, page 163:

Revise Table 15.4.2 as shown below:

TABLE 15.4-2 SEISMIC COEFFICIENTS FOR NONBUILDING STRUCTURES NOT SIMILAR TO BUILDINGS

Nonbuilding Structure Type	Detailing Requirements[c]	R	Ω_0	C_d	Structural System And Height Limits (ft.)[a,d]				
					A & B	C	D	E	F
Elevated tanks, vessels, bins, or hoppers:									
On symmetrically braced legs (not similar to buildings)	15.7.10	3	2[b]	2.5	NL	NL	160	100	100
On unbraced legs or asymmetrically braced legs (not similar buildings)	15.7.10	2	2[b]	2.5	NL	NL	100	60	60
Single pedestal or skirt supported									
- welded steel	15.7.10	2	2[b]	2	NL	NL	NL	NL	NL
- welded steel with special detailing	15.7.10 and 15.7.10.5 a and b.	3	2[b]	2	NL	NL	NL	NL	NL
- prestressed or reinforced concrete	15.7.10	2	2[b]	2	NL	NL	NL	NL	NL
- prestressed or reinforced concrete with special detailing	15.7.10 and ~~14.2.3.6~~ ACI 318, Chapter 21 Sections 21.2 and 21.7	3	2[b]	2	NL	NL	NL	NL	NL

Chapter C6, page 291:

Revise Eq. C6-8 as shown below:

$$\Delta K = \left(K_{33,u} - K_{33,d}\right)\frac{K_{zd}}{K_{33,d}}F_{\Delta K}\left(x\right) \qquad \text{(C6-8)}$$

$\left|\Delta K\right| \leq \left|K_{zu} - K_{zd}\right|$ *(The "=" is replaced by "≤")*

Chapter C6, page 292:

Revise Eq. C6-10 as shown below:

$$x_0 = c_3 \times 10^{-(K_{33,d}-K_{33,u})^2-2.3} \qquad \text{(C6-10)}$$

(The constant c_3 needs to be added)

Chapter C7, page 332:

Revise References as shown below:

[Ref. C7-43] Tobiasson, W. (Apr. 1999). "Discussion of Ref. ~~C7-5642~~ C7-42. *J. Struct. Engrg. (ASCE)*, 125(4), 470–471.

[Ref. C7-55] ~~Isyumou~~ Isyumov, N., and Mikitiuk, M. (June 1992). "Wind tunnel modeling of snow accumulation on large roofs." In *Proc. 2nd International Conf. Snow Engrg*. Santa Barbara, CA.

Chapter C8, page 337:

Revise Section C8.3, (C8-1) as shown below:

The flow rate through a single drainage system is as follows:

~~$Q = 0.0104A_i$~~ $Q = 0.0104\ A\ i$ (C8-1)

~~(in SI: $Q = 0.278 \times 10^{-6}\ A_i$)~~ (in SI: $Q = 0.278 \times 10^{-6}\ A\ i$)

(The symbol "i" is rainfall intensity, not a subscript to "A")

Chapter C8, page 338:

Revise Section C8.3, Example 1, (C8-1) as shown below:

Flow rate, Q, for the secondary drainage 4 in. diameter (102 mm) roof drain:

~~$Q = 0.0104A_i$~~ $Q = 0.0104\ A\ i$ (C8-1)

$Q = 0.0104\ (2500)(3.75) = 97.5$ gal/min ($0.0062\ m^3/s$)

Chapter C8, page 338:

Revise Section C8.3, Example 2, (C8-1) as shown below:

Flow rate, Q, for the secondary drainage, 12 in. (305 mm) wide channel scupper:

~~$Q = 0.0104\ A_i$~~ $Q = 0.0104\ A\ i$ (C8-1)

$Q = 0.0104(11{,}500)(1.5) = 179$ gal/min ($0.0113\ m^3/s$)

Errata Posted on October 3, 2006

Chapter 12, page 129:

Revise Section 12.8.2 as shown below:

12.8.2 Period Determination. The fundamental period of the structure, T, in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period, T, shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 12.8-1 and the approximate fundamental period, T_a, determined ~~from Eq. 12.8-7.~~ in accordance with Section 12.8.2.1. As an alternative to performing an analysis to determine the fundamental period, T, it is permitted to use the approximate building period, T_a, calculated in accordance with Section 12.8.2.1, directly.

Chapter 14, page 156:

Revise Section 14.2.2.7 as shown below:

14.2.2.17 General Requirements for Anchoring to Concrete. Modify Section D.3.3 by deleting Sections ~~D.3.2~~ D.3.3.2 through D.3.3.5~~.2~~ and replace with the following: *(remainder of Section is unchanged).*

Chapter 15, Table 15.4-2, page 163:

Revise Table 15.4-2 as shown below:

Tanks or vessels supported on structural towers similar to buildings	15.5.5	Use values for the appropriate structure type in the categories for building frame systems and moment resisting frame systems listed in Table 12.2-1 or Table 15.4-1.

Chapter 15, page 166-167:

Revise Items 1 and 2 of Part c of Section 15.7.2 as shown below by removing the Importance Factor:

c. Vertical earthquake forces shall be considered in accordance with the applicable reference document. If the reference document permits the user the option of including or excluding the vertical earthquake force, to comply with this standard, it shall be included. For tanks and vessels not covered by a reference document, the forces due to the vertical acceleration shall be defined as follows:

(1) Hydrodynamic vertical and lateral forces in tank walls: The increase in hydrostatic pressures due to the vertical excitation of the contained liquid shall correspond to an effective increase in unit weight, γ_L, of the stored liquid equal to ~~$0.2S_{DS}I\gamma_L$.~~ $0.2S_{DS}\gamma_L$.

(2) Hydrodynamic hoop forces in cylindrical tank walls: In a cylindrical tank wall, the hoop force per unit height, N_h, at height y from the base, associated with the vertical excitation of the contained liquid, shall be computed in accordance with Eq. 15.7-1.

$$N_h = 0.2S_{DS}\gamma_L(H_L - y)\left(\frac{D_i}{2}\right) \qquad (15.7\text{-}1)$$

where

D_i = inside tank diameter

H_L = liquid height inside the tank

y = distance from base of the tank to height being investigated

γ_L = unit weight of stored liquid

(3) Vertical inertia forces in cylindrical and rectangular tank walls: Vertical inertia forces associated with the vertical acceleration of the structure itself shall be taken equal to ~~$0.2S_{DS}IW$.~~ $0.2S_{DS}W$.

Chapter 23, page 237:

Revise the citation of AWWA D100 as shown below:

AWWA D100
Sections 15.4.1, 15.7.7.1, 15.7.9.4, 15.7.10.6, 15.7.10.6.2
Welded Steel Tanks for Water Storage, ~~2005~~ 2006

Revise the citation of ICC-ES AC 156-04 as shown below:

***ICC-ES AC 156-04**
Section 13.2.5
~~*Acceptance Criteria for Seismic Qualification Testing of Nonstructural Components*, 2000~~
Seismic Qualification by Shake-table Testing of Nonstructural Components and Systems, 2004

Revise the citation of NFPA 59A as shown below:

NFPA 59A
Section 15.4.8
Production, Storage, and Handling of Liquefied Natural Gas (LNG), ~~2005~~ 2006

Chapter C22, page 383: Revise as shown below:

C22 SEISMIC GROUND MOTION MAPS

The 2005 edition of ASCE 7 continues to utilize spectral response seismic design maps that reflect seismic hazards on the basis of contours. These maps were developed by the United States Geological Survey (USGS) and were updated for the 2005 edition. The USGS ~~has also developed~~ is developing a companion software program that calculates spectral values for a specific site based on a site's longitude, latitude, and site soil classification. The calculated values are based on the data used to prepare the maps in Section 22.0. The spectral values may be adjusted for Site Class effects using the Site Classifications Procedure in Section 20.0 and the site coefficients in Section 11.4. Longitude and latitude for a given address can be found at a variety of Web sites. The software program should be used for establishing spectral values for design because the maps found in ASCE 7 and at Web sites are at too large a scale to provide accurate spectral values for most sites. Upon its completion, the ~~The~~ software program will be included ~~is available~~ on the CD-ROM version of ASCE 7-05, and it may also be accessed at the USGS Web site at eqhazmaps.usgs.gov, or through the SEI Web site at www.seinstitute.org Upon its completion, the availability of the software program will be widely advertised however, as of this writing, it is unknown when the program will be available.

Errata Posted on September 15, 2006

Chapter 6 – page 27:

In the second equation for R_ℓ on the right hand side of the equation for etta the "E" should not be in the equation as shown below:

As printed: $R_\ell = R_B$ setting $\eta = 4.6 n_1 E B / \overline{V}_{\bar{z}}$

Correct equation: $R_\ell = R_B$ setting $\eta = 4.6 n_1 B / \overline{V}_{\bar{z}}$

Chapter 12:

Table 12.2-1: Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force Resisting System		ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R[a]	System Overstrength Factor, Ω_O[g]	Deflection Amplification Factor, C_d[b]	Structural System Limitations and Building Height (ft) Limit[c]				
						Seismic Design Category				
						B	C	D[d]	E[d]	F[e]
B11 Printed	Composite ~~Ordinary~~ steel and concrete eccentrically braced frames	14.3	8	2	4	NL	NL	160	160	100
B11 Corrected	Composite ~~Ordinary~~ steel and concrete eccentrically braced frames	14.3	8	2 ½	4	NL	NL	160	160	100
C3 Printed	Intermediate Steel Moment Frames	12.2.5.6, 12.2.5.7, 12.2.5.8, 12.2.5.9, and 14.1	4.5	3	4	NL	NL	35[h, i]	NP[h]	NP[i]
C3 Corrected In Red	Intermediate Steel Moment Frames	12.2.5.6, 12.2.5.7, 12.2.5.8, 12.2.5.9, and 14.1	4.5	3	4	NL	NL	35[h]	NP[h]	NP[i]
G2 Printed	Intermediate steel moment frames	14.1	1 ½	1 ¼	1 ½	35	35	35[h]	NP[h, i]	NP[h, i]
G2 Corrected In Red	Intermediate steel moment frames	14.1	1 ½	1 ¼	1 ½	35	35	35[h]	NP[h]	NP[i]
G3 Printed	Ordinary steel moment frames	14.1	1 ¼	1 ¼	1 ¼	35	35	NP	NP[h, i]	NP[h, i]
G3 Corrected In Red	Ordinary steel moment frames	14.1	1 ¼	1 ¼	1 ¼	35	35	NP[h]	NP[h]	NP[i]

System B11 – Ω_O is 2½ not 2; The change in BLUE reflects an error in the original posting of this errata – not an error in the printed document. Our apologies.
System C3 – remove footnote i for SDC D;
System G2 – remove footnote i for SDC E and remove footnote h for SDC F;
System G3 – add footnote h for SDC D, remove footnote i for SDC E and remove footnote h for SDC F.

Section 12.4.2.3 – page 127, Basic Combinations for Allowable Stress Design (See Section 2.4.1 and 2.2 for notation).

6. $(1.0 + 0.105_{DS})\ D + H + F + 0.525\ \rho\ Q_E + 0.75\ L + 0.75\ (L_r$ or S or R)
6. $(1.0 + 0.105\ S_{DS})\ D + H + F + 0.525\ \rho\ Q_E + 0.75\ L + 0.75\ (L_r$ or S or R)

(*The S is missing in the first set of parentheses.)*

Table 12.6-1 – page 128:

D, E, F	Irregular structures with $T < 3.5\ T_s$ and having only horizontal irregularities type 2, 3, 4, or 5 of Table ~~12.2-1~~ 12.3-1 or vertical irregularities type 4, 5a or 5b of Table ~~12.3-1~~ 12.3-2	P	P	P

Chapter 13:

Section 13.1.4, page 143:

Modify Exceptions 4 and 5 as shown below:

4. Mechanical and electrical components in Seismic Design Categories D, E, or ~~and~~ F where the component importance factor, I_p, is equal to 1.0 and ~~either~~ both of the following conditions apply:

   a. flexible connections between the components and associated ductwork, piping, and conduit are provided, ~~or~~ and
   b. components are mounted at 4 ft (1.22 m) or less above a floor level and weigh 400 lb (1780 N) or less.

5. Mechanical and electrical components in Seismic Design Categories D, E, and F where the component importance factor, I_p, is equal to 1.0 and both of the following conditions apply:

   a. Flexible connections between the components and associated ductwork, piping, and conduit are provided, and
   b. The components weigh 20 lb (89 N) or less or, for distribution systems, weighing 5 lb/ft (73 N/m) or less.

Section 13.4.2, page 145:

The word "or" should be added to the second listing of items as follows:

13.4.2 Anchors in Concrete or Masonry. Anchors embedded in concrete or masonry shall be proportioned to carry the least of the following:

a. 1.3 times the force in the component and its supports due to the prescribed forces.
b. The maximum force that can be transferred to the anchor by the component and its supports.

The value of R_p used in Section 13.3.1 to determine the forces in the connected part shall not exceed 1.5 unless

a. The component anchorage is designed to be governed by the strength of a ductile steel element or
b. The design of post-installed anchors in concrete used for the component anchorage is prequalified for seismic applications in accordance with ACI 355.2 or
c. The anchor is designed in accordance with Section 14.2.2.17.

Chapter 15 – page 169:

Table 15.7-3: The heading of the third column of Occupancy Category should be “IV” not “III”

Chapter 23 – page 236:

API
American Petroleum Institute
1220 L Street
Washington, D.C. 20005-4070

API 650
Sections 15.4.1, 15.7.8.1, 15.7.9.4
Welded Steel Tanks For Oil Storage, 10th Edition, Addendum 4, ~~2005~~ 2006.

ASCE/SEI
American Society of Civil Engineers
Structural Engineering Institute
1801 Alexander Bell Drive
Reston, VA 20191-4400

ASCE 19
Section 14.1.1, 14.1.7
Structural Applications for Steel Cables for Buildings, ~~2002~~ 1996.

Chapter C5 – Section C5.4.5 – “Duration of Impact”, page 278:

The equation reference in last sentence should be changed from C5-2 to C5-3 as follows:

The recommended value for use in Eq. ~~C5-2~~ C5-3 is therefore 0.03 s.

Errata Posted on January 6, 2006

Please see below for errata posted after this date

Chapter 6 – Figure 6-18A, page 66:

In the heading, upper right hand corner, the “q” should be “θ” so that the heading reads:

Monoslope Free Roofs
$\theta \leq 45°$, $\gamma = 0°$, 180°

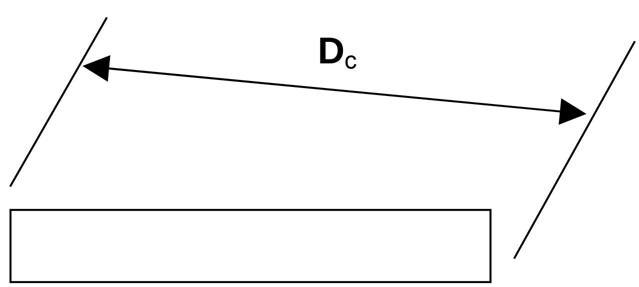
Chapter 6 – Figure 6-18B, page 67:

In the heading of the table for Wind Direction, the “g” should be “γ” so that the heading reads:

Wind Direction, $\gamma = 0°$, 180°

Chapter 10 – Figure 10-1, page 102:

The dimension D_c in the upper right-hand diagram is measured from the upper corner of the rectangle on the left side to the lower corner on the right side as shown below:



Chapter 11 – Section 11.6, page 116:

Modify the first sentence as follows:

> Structures shall be assigned a Seismic Design Category in accordance with this Section. ~~Section 11.6.1.1.~~

Chapter 12 – Table 12.8-2, page 129:

Remove two horizontal lines and indent two headings for clarity. Properly formatted table is presented below:

Table 12.8-2 Values of Approximate Period Parameters C_t and x

Structure Type	C_t	x
Moment resisting frame systems in which the frames resist 100 percent of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel Moment Resisting Frames	0.028 (0.0724)[a]	0.8
Concrete Moment Resisting Frames	0.016 (0.0466)[a]	0.9
Eccentrically braced steel frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

[a]- metric equivalents are shown in parentheses

Chapter 12 – Section 12.8.4.2, page 130:

Remove the units in two places in the first paragraph as shown below:

12.8.4.2 Accidental Torsion. Where diaphragms are not flexible, the design shall include the inherent torsional moment (M_t) ~~(kip or kN)~~ resulting from the location of the structure masses plus the accidental torsional moments (M_{ta}) ~~(kip or kN)~~ caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces.

Where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass need not be applied in both of the orthogonal directions at the same time, but shall be applied in the direction that produces the greater effect.

Chapter 12 – Section 12.14.1.1, page 137:

Revise item 8 as follows:

9. For buildings with a diaphragm that is not flexible, the distance between the center of rigidity and the center of mass parallel to each major axis shall not exceed 15 percent of the greatest width of the diaphragm parallel to that axis. In addition, the following <u>two equations</u> shall be satisfied ~~for each major axis direction~~:

$$\sum_{i=1}^{m} k_{1i} d_i^2 + \sum_{j=1}^{n} k_{2j} d_j^2 \geq 2.5(0.05 + \frac{e_1}{b_1}) b_1^{\,2} \sum_{i=1}^{m} k_{1i} \qquad \text{(Eq. 12.14-2}\underline{\text{A}}\text{)}$$

$$\underline{\sum_{i=1}^{m} k_{1i} d_i^2 + \sum_{j=1}^{n} k_{2j} d_j^2 \geq 2.5(0.05 + \frac{e_2}{b_2}) b_2^{\,2} \sum_{i=1}^{m} k_{1i}} \qquad \underline{\text{(Eq. 12.14-2B)}}$$

where (see Figure 12.14-1):

k_{1i} is the lateral load stiffness of wall "*i*" or braced frame "*i*" parallel to major axis 1
k_{2j} is the lateral load stiffness of wall "*j*" or braced frame "*j*" parallel to major axis 2
d_{1i} is the distance from the wall "*i*" or braced frame "*i*" to the center of rigidity, perpendicular to major axis 1
d_{2j} is the distance from the wall "*j*" or braced frame "*j*" to the center of rigidity, perpendicular to major axis 2
e_1 is the distance perpendicular to major axis 1 between the center of rigidity and the center of mass
b_1 is the width of the diaphragm perpendicular to major axis 1
<u>e_2 is the distance perpendicular to major axis 2 between the center of rigidity and the center of mass</u>
<u>b_2 is the width of the diaphragm perpendicular to major axis 2</u>
m is the number of walls and braced frames resisting lateral force in direction 1
n is the number of walls and braced frames resisting lateral force in direction 2

Eqs. 12.14-2<u>A and B</u> need not be checked where a structure fulfills all the following limitations:

1. The arrangement of walls or braced frames is symmetric about each major axis direction,
2. The distance between the two most separated lines of walls or braced frames is at least 90 percent of the dimension of the structure perpendicular to that axis direction, and
3. The stiffness along each of the lines considered for item 2 above is at least 33 percent of the total stiffness in that axis direction.

Chapter 13 – Section 13.4.2, page 145:

The referenced section in item c should be 14.2.2.17 rather than 14.2.2.14 as follows:

c. The anchor is designed in accordance with Section ~~14.2.2.14~~ <u>14.2.2.17</u>.

Chapter C5 – Section C5.4.5 – Equation C5-3, page 278:

The denominator should be " 2g Δt " rather than " 2g Δg "

Chapter C Appendix C: The commentary on Appendix C, Serviceability Considerations was inadvertently omitted from the printed version of ASCE 7-05 and is provided below. We apologize for this omission.

Chapter C Appendix C
Serviceability Considerations

CC. Serviceability Considerations

Serviceability limit states are conditions in which the functions of a building or other structure are impaired because of local damage, deterioration or deformation of building components, or because of occupant discomfort. While safety generally is not an issue with serviceability limit states, they nonetheless may have severe economic consequences. The increasing use of the computer as a design tool, the use of stronger (but not stiffer) construction materials, the use of lighter architectural elements, and the uncoupling of the nonstructural elements from the structural frame, may result in building systems that are relatively flexible and lightly damped. Limit states design emphasizes that serviceability criteria (as they always have been) are essential to ensure functional performance and economy of design for such building structural systems [Refs. CC-1, CC-2, CC-3].

There are three general types of unserviceability that may be experienced:

1. Excessive deflections or rotation that may affect the appearance, functional use or drainage of the structure, or may cause damaging transfer of load to non-load supporting elements and attachments.
2. Excessive vibrations produced by the activities of building occupants, mechanical equipment, or the wind, which may cause occupant discomfort or malfunction of building service equipment.
3. Deterioration, including weathering, corrosion, rotting, and discoloration.

In checking serviceability, the designer is advised to consider appropriate service loads, the response of the structure, and the reaction of the building occupants.

Service loads that may require consideration include static loads from the occupants and their possessions, snow or rain on roofs, temperature fluctuations, and dynamic loads from human activities, wind-induced effects, or the operation of building service equipment. The service loads are those loads that act on the structure at an arbitrary point in time. (In contrast, the nominal loads have a small probability of being exceeded in any year; factored loads have a small probability of being exceeded in 50 years.) Appropriate service loads for checking serviceability limit states may be only a fraction of the nominal loads.

The response of the structure to service loads normally can be analyzed assuming linear elastic behavior. However, members that accumulate residual deformations under service loads may require examination with respect to this long-term behavior. Service loads used in analyzing creep or other long-term effects may not be the same as those used to analyze elastic deflections or other short-term or reversible structural behavior.

Serviceability limits depend on the function of the building and on the perceptions of its occupants. In contrast to the ultimate limit states, it is difficult to specify general serviceability limits that are applicable to all building structures. The serviceability limits presented in Sections CC.1.1, CC.1.2, and CC.1.3 provide general guidance and have usually led to acceptable performance in the past. However, serviceability limits for a specific building should be determined only after a careful analysis by the engineer and architect of all functional and economic requirements and constraints in conjunction with the building owner. It should be recognized that building occupants are able to perceive structural deflections, motion, cracking, or other signs of possible distress at levels that are much lower than those that would indicate that structural failure was impending. Such signs of distress may be taken incorrectly as an indication that the building is unsafe and diminish its commercial value.

CC.1.1 Vertical Deflections Excessive vertical deflections and misalignment arise primarily from three sources: (1) gravity loads, such as dead, live, and snow loads; (2) effects of temperature, creep, and differential settlement; and (3) construction tolerances and errors. Such deformations may be visually objectionable, may cause separation, cracking, or leakage of exterior cladding, doors, windows and seals, and may cause damage to interior components and finishes. Appropriate limiting values of deformations depend on the type of structure, detailing, and intended use [Ref. CC-4]. Historically, common deflection limits for horizontal members have been 1/360 of the span for floors subjected to full nominal live load and 1/240 of span for roof members. Deflections of about 1/300 of the span (for cantilevers, 1/150 of length) are visible and may lead to general architectural damage or cladding leakage. Deflections greater than 1/200 of the span may impair operation of moveable components such as doors, windows, and sliding partitions.

In certain long-span floor systems, it may be necessary to place a limit (independent of span) on the maximum deflection to minimize the possibility of damage of adjacent nonstructural elements [Ref. CC-5]. For example, damage to nonload-bearing partitions may occur if vertical deflections exceed more than about 10 mm (3/8 in.) unless special provision is made for differential movement [Ref. CC-6]; however, many components can and do accept larger deformations.

Load combinations for checking static deflections can be developed using first-order reliability analysis [Ref. CC-4]. Current static deflection guidelines for floor and roof systems are adequate for limiting surficial damage in most buildings. A combined load with an annual probability of 0.05 of being exceeded would be appropriate in most instances. For serviceability limit states involving visually objectionable deformations, repairable cracking or other damage to interior finishes, and other short-term effects, the suggested load combinations are:

$$D + L \qquad \text{(CC-1a)}$$

$$D + 0.5S \qquad \text{(CC-1b)}$$

For serviceability limit states involving creep, settlement, or similar long-term or permanent effects, the suggested load combination is:

$$D + 0.5L \qquad \text{(CC-2)}$$

The dead load effect, D, used in applying Equations CC.1 and CC.2 may be that portion of dead load that occurs following attachment of nonstructural elements. Live load, L, is defined in Chapter 4. For example, in composite construction, the dead load effects frequently are taken as those imposed after the concrete has cured; in ceilings, the dead load effects may include only those loads placed after the ceiling structure is in place.

CC.1.2 Drift of Walls and Frames Drifts (lateral deflections) of concern in serviceability checking arise primarily from the effects of wind. Drift limits in common usage for building design are on the order of 1/600 to 1/400 of the building or story height [Ref. CC-7]. These limits generally are sufficient to minimize damage to cladding and nonstructural walls and partitions. Smaller drift limits may be appropriate if the cladding is brittle. An absolute limit on interstory drift may also need to be imposed in light of evidence that damage to non-structural partitions, cladding and glazing may occur if the interstory drift exceeds about 10 mm (3/8 in.) unless special detailing practices are made to tolerate movement [Refs. CC-6, CC-8]. Many components can accept deformations that are significantly larger.

Use of the factored wind load in checking serviceability is excessively conservative. The load combination with an annual probability of 0.05 of being exceeded, which can be used for checking short-term effects, is

$$D + 0.5L + 0.7W \qquad \text{(CC-3)}$$

obtained using a procedure similar to that used to derive Eqs. CC-1a and CC-1b. Wind load, W, is defined in Chapter 6. Due to its transient nature, wind load need not be considered in analyzing the effects of creep or other long-term actions.

Deformation limits should apply to the structural assembly as a whole. The stiffening effect of nonstructural walls and partitions may be taken into account in the analysis of drift if substantiating information regarding their effect is available. Where load cycling occurs, consideration should be given to the possibility that increases in residual deformations may lead to incremental structural collapse.

CC.1.3 Vibrations Structural motions of floors or of the building as a whole can cause the building occupants discomfort. In recent years, the number of complaints about building vibrations has been increasing. This increasing number of complaints is associated in part with the more flexible structures that result from modern construction practice. Traditional static deflection checks are not sufficient to ensure that annoying vibrations of building floor systems or buildings as a whole will not occur [Ref. CC-1]. While control of stiffness is one aspect of serviceability, mass distribution and damping are also important in controlling vibrations. The use of new materials and building systems may require that the dynamic response of the system be considered explicitly. Simple dynamic models often are sufficient to determine whether there is a potential problem and to suggest possible remedial measurements [Refs. CC-9, CC-10].

Excessive structural motion is mitigated by measures that limit building or floor accelerations to levels that are not disturbing to the occupants or do not damage service equipment. Perception and tolerance of individuals to vibration is dependent on their expectation of building performance (related to building occupancy) and to their level of activity at the time the vibration occurs [Ref. CC-11]. Individuals find continuous vibrations more objectionable than transient vibrations. Continuous vibrations (over a period of minutes) with acceleration on the order of 0.005 g to 0.01 g are annoying to most people engaged in quiet activities, whereas those engaged in physical activities or spectator events may tolerate steady-state accelerations on the order of 0.02 g to 0.05 g. Thresholds of annoyance for transient vibrations (lasting only a few seconds) are considerably higher and depend on the amount of structural damping present [Ref. CC-12]. For a finished floor with (typically) 5 percent damping or more, peak transient accelerations of 0.05 g to 0.1 g may be tolerated.

Many common human activities impart dynamic forces to a floor at frequencies (or harmonics) in the range of 2 to 6 Hz [Refs. CC-13 through CC-16]. If the fundamental frequency of vibration of the floor system is in this range and if the activity is rhythmic in nature (e.g., dancing, aerobic exercise, cheering at spectator events), resonant amplification may occur. To prevent resonance from rhythmic activities, the floor system should be tuned so that its natural frequency is well removed from the harmonics of the excitation frequency. As a general rule, the natural frequency of structural elements and assemblies should be greater than 2.0 times the frequency of any steady-state excitation to which they are exposed unless vibration isolation is provided. Damping is also an effective way of controlling annoying vibration from transient events, as studies have shown that individuals are more tolerant of vibrations that damp out quickly than those that persist [Ref. CC-12].

Several recent studies have shown that a simple and relatively effective way to minimize objectionable vibrations to walking and other common human activities is to control the floor stiffness, as measured by the maximum deflection independent of span. Justification for limiting the deflection to an absolute value rather than to some fraction of span can be obtained by considering the dynamic characteristics of a floor system modeled as a uniformly loaded simple span. The fundamental frequency of vibration, f_o, of this system is given by

$$f_o = \frac{\pi}{2l^2}\sqrt{\frac{EI}{\rho}} \qquad \text{(CC-4)}$$

in which EI = flexural rigidity of the floor, l = span, and ρ = w/g = mass per unit length; g = acceleration due to gravity (9.81 m/s^2), and w = dead load plus participating live load. The maximum deflection due to w is

$$\delta = (5/384)(wl^4/EI) \qquad \text{(CC-5)}$$

Substituting EI from this equation into Eq. CC-3, we obtain

$$f_o \approx 18/\sqrt{\delta} \quad (\delta \ \ in \ \ mm) \qquad \text{(CC-6)}$$

This frequency can be compared to minimum natural frequencies for mitigating walking vibrations in various occupancies [Ref. CC-17]. For example, Eq. CC-6 indicates that the static deflection due to uniform load, w, must be limited to about 5 mm, independent of span, if the fundamental frequency of vibration of the floor system is to be kept above about 8 Hz. Many floors not meeting this guideline are perfectly serviceable; however, this guideline provides a simple means for identifying potentially troublesome situations where additional consideration in design may be warranted.

CC.2 Design for Long-Term Deflection Under sustained loading, structural members may exhibit additional time-dependent deformations due to creep, which usually occur at a slow but persistent rate over long periods of time. In certain applications, it may be necessary to limit deflection under long-term loading to specified levels. This can be done by multiplying the immediate deflection by a creep factor, as provided in material standards, that ranges from about 1.5 to 2.0. This limit state should be checked using load combination CC.2.

CC.3 Camber Where required, camber should be built into horizontal structural members to give proper appearance and drainage and to counteract anticipated deflection from loading and potential ponding.

CC.4 Expansion and Contraction Provisions should be made in design so that if significant dimensional changes occur, the structure will move as a whole and differential movement of similar parts and members meeting at joints will be a minimum. Design of expansion joints to allow for dimensional changes in portions of a structure separated by such joints should take both reversible and irreversible movements into account. Structural distress in the form of wide cracks has been caused by restraint of thermal, shrinkage, and prestressing deformations. Designers are advised to provide for such effects through relief joints or by controlling crack widths.

CC.5 Durability Buildings and other structures may deteriorate in certain service environments. This deterioration may be visible upon inspection (weathering, corrosion, staining) or may result in undetected changes in the material. The designer should either provide a specific amount of damage tolerance in the design or should specify adequate protection systems and/or planned maintenance to minimize the likelihood that such problems will occur. Water infiltration through poorly constructed or maintained wall or roof cladding is considered beyond the realm of designing for damage tolerance. Waterproofing design is beyond the scope of this standard. For portions of buildings and other structures exposed to weather, the design should eliminate pockets in which moisture can accumulate.

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[CC-8] Freeman, S. (1977). "Racking tests of high rise building partitions." *J. Str. Div., ASCE* 103(8): 1673–1685.

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[CC-15] Allen, D.E. (1990a). "Floor vibrations from aerobics." *Canadian J. Civil Engr*. 19(4): 771–779.

[CC-16] Allen, D.E. (1990b). "Building vibrations from human activities." *Concrete International* 12(6): 66–73.

[CC-17] Allen, D.E. and Murray, T.M. (1993). "Design criterion for vibrations due to walking." *Engineering Journal, AISC* 30(4):117–129.

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