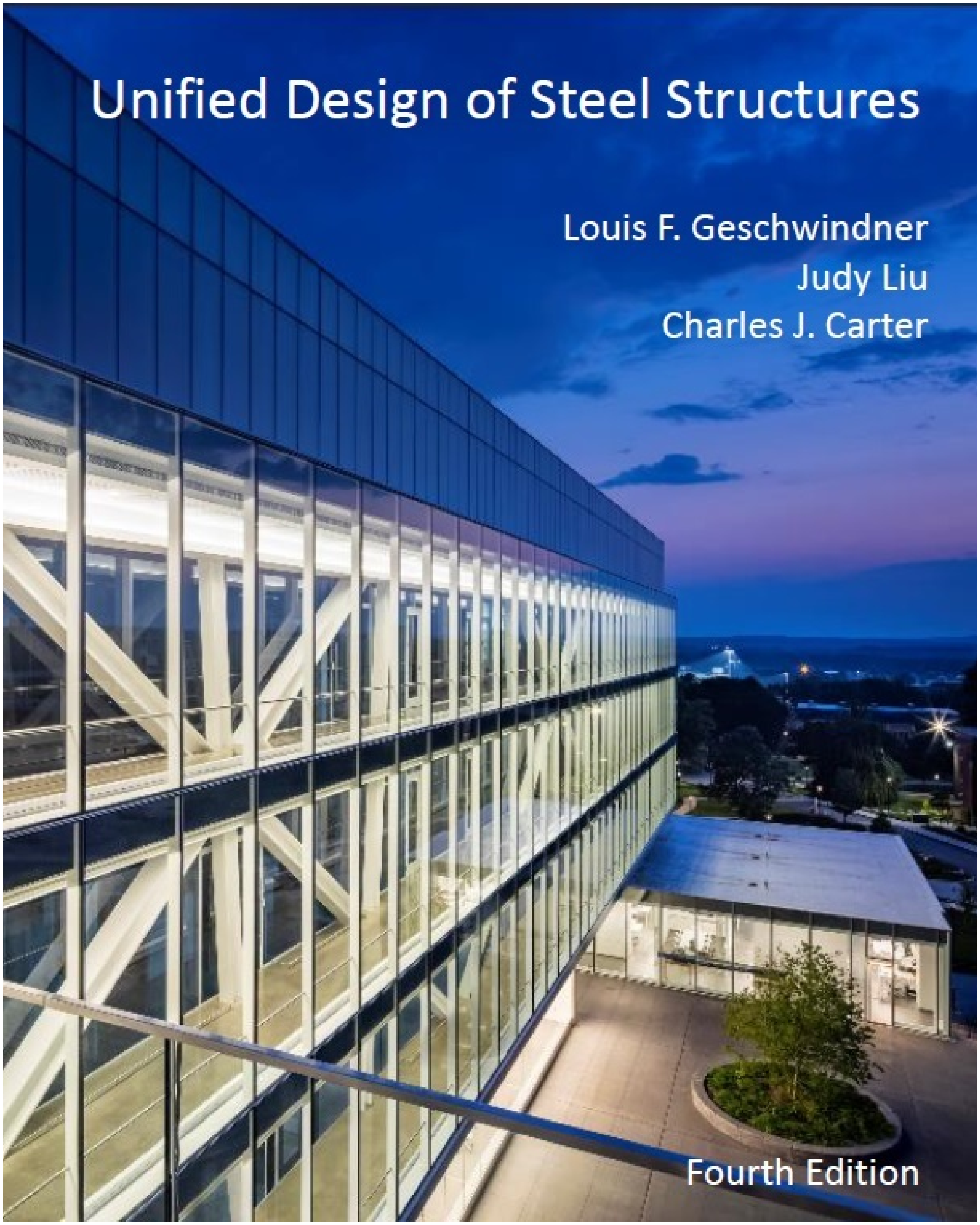

Selected Homework Problem Answers

Unified Design of Steel Structures, 4th Edition

Chapter 1 Selected Answers

1. Where could one find information about the provisions of the 1961 AISC *Specification*?

For the answer, see Section 1.2

3. Which chapter of the AISC *Specification* provides information about:

a. general requirements for analysis and design
b. design of members for flexure
c. design of connections
For the answer, see Section 1.2

5. What types of members are addressed in AISC *Specification* Chapter I?

For the answer, see Section 1.2

7. What types of members does Part 5 of the *Manual* address?

For the answer, see Section 1.3

9. List and define the three basic goals of a design team for the design of any building.

For the answer, see Section 1.5

11. What is the difference between beams, girders, and joists?

For the answer, see Section 1.6

13. What type of structural system uses the combined properties of two or more different types of materials to resist the applied loads?

For the answer, see Section 1.7, Section 1.2

15. In designing a steel structure, what must be the primary concern of the design engineer?

For the answer, see Section 1.8

17. Describe the difference between a strength limit state of a structure and a serviceability limit state.

For the answer, see Section 1.8

19. Provide a brief description of plastic design (PD).

For the answer, see Section 1.11

21. Define the terms in Equation 1.2.

For the answer, see Section 1.12

Chapter 1 Selected Answers, cont'd.

23. It was shown that designs by ASD and LRFD result in the same reliability index for a specific Live-to-Dead load ratio. What is that ratio and describe the ramifications for other Live-to-Dead load ratios.

For the answer, see Section 1.12

25. Provide three examples of serviceability limit states.

For the answer, see Section 1.13

Chapter 2 Selected Answers

1. Name and describe five basic types/sources of building loads.

For the answer, see Section 2.2

3. What is one source you can consult to find the snow load data for a particular region as well as maps showing wind gust data to allow you to calculate wind loads?

For the answer, see Sections 2.2, 2.3

5. Give a source you can use to find the minimum uniformly distributed live load for buildings.

For the answer, see Section 2.3

7. What is the rationale for the live load reduction factor, when can this reduction factor be used, and what are the limitations on its use?

For the answer, see Section 2.3.2

9. Identify and briefly describe three factors that affect the magnitude of the snow load on an unobstructed flat roof.

For the answer, see Section 2.3.3

11. If the local building code specifies a design load that differs from what is stated in ASCE 7, which document should you follow?

For the answer, see Section 2.3

13. If a response modification factor of 3 is chosen in the design of a steel building to resist seismic loads, what design specification should be consulted?

For the answer, see Section 2.3.5

15. Strength load combinations that are incorporated by the LRFD method take into account what two factors?

For the answer, see Section 2.4

Chapter 2 Selected Answers, cont'd.

17. Using ASCE 7, determine the minimum uniformly distributed live load for library stacks.

For the answer, see Table 2.2 or ASCE 7 Table 4.3-1

19. Using ASCE 7, determine the nominal uniformly distributed self-weight of a 6 in. thick reinforced concrete slab. If the slab has a 7/8 in. hardwood floor topping, what would its self-weight be?

Use Tables C3.1-2 and C3.1-1a to obtain 79 psf

21. If a design is carried out according to the applicable building codes and standards, is it guaranteed that a failure will not occur? Explain your answer.

For the answer, see Section 2.7

23. If the framing plan shown in Figure P2.23 were for the roof of a structure that carried a dead load of 55 psf and a roof live load of 30 psf, determine the required moment and shear strength for beams and girders, and axial strength for columns, as required below for (a) design by LRFD and (b) design by ASD. Do not reduce the roof live loads.

i. The girder on column line A between column lines 1 and 2 if the deck spans from line A-A to B-B.

ii. The beam on column line 3 between column lines B and C if the deck spans from line 2-2 to 3-3 to 4-4.

iii. The column at the corner on lines 1 and E.

iv. The column on the edge at the intersection of lines 1 and B.

v. The interior column at the intersection of column lines C and 2.

Part a. uniform load for LRFD $w_u = 1.2(55) + 1.6(30) = 114 \text{ psf}$

Part b. uniform load for ASD $w_a = 55 + 30 = 85 \text{ psf}$

No Roof Live Load Reductions

	Part a. LRFD	**Part b. ASD**
i.	M_u = 111 ft-kips V_u = 17.8 kips	M_a = 83.0 ft-kips V_a = 13.3 kips
ii.	M_u = 223 ft-kips V_u = 35.6 kips	M_a = 166 ft-kips V_a = 26.6 kips
iii.	P_u = 17.8 kips	P_a = 13.3 kips
iv.	P_u = 35.6 kips	P_a = 26.6 kips
v.	P_u = 71.3 kips	P_a = 53.1 kips

Chapter 2 Selected Answers, cont'd.

25. The 18-story building given in Figure P2.24 must support a floor and roof dead load of 80 psf, a floor live load of 50 psf, and a roof live load of 30 psf as given in problem 24. The building has a uniform story heights of 14 ft. The wind load is determined to be 40 psf over the upper 50 ft, and the lateral load is to be resisted in each direction by perimeter moment frames. The 4-bay moment frames are located on grid lines 1 and 8 and the 7-bay moment frames are located along lines A and E. Use the load case that includes dead, live and wind loads to calculate resultant wind forces in each frame at the roof level and at the next level down for (a) design by LRFD and (b) design by ASD.

(a) LRFD, $1.2D + 1.0W + L + 0.5L_r$

(partial answer) on each 7-bay frame, $P_{u\text{-}roof}$ = 13.9 kips, $P_{u\text{-}floor}$ = 27.7 kips

(b) ASD, $D + 0.75L + 0.75(0.6W) + 0.75L_r$

(partial answer) on each 7-bay frame, $P_{a\text{-}roof}$ = 6.24 kips, $P_{a\text{-}floor}$ = 12.5 kips

Chapter 3 Selected Answers

1. When was the first AISC *Specification* published and what was its purpose?

For the answer, see Section 3.2

3. Sketch and label a typical stress-strain curve for steel subjected to a simple uniaxial tension test.

For the answer, see Section 3.3; base on Figure 3.5

5. What happens to a steel element when it is loaded beyond the elastic limit and then unloaded?

For the answer, see Section 3.3.

7. Sketch and label 10 different structural shape cross sections whose properties are given in the AISC *Manual*.

For the answer, see Section 3.4; base on Figure 3.7 and the AISC Manual

9. What are the nominal and actual depths of a W16×57 wide-flange member? What is the weight of this member per linear foot? (Hint: Use your AISC *Manual*.)

(Partial answer) from Manual Table 1-1, the actual depth is 16.4 in.

11. What are the nominal and actual depths of a W14×808 wide-flange member? Compare these to the nominal and actual depths of a W14×159. (Hint: Use your AISC *Manual*.)
(Partial answer) from Manual Table 1-1, for the W14×808, the actual depth is 22.8 in.; for the W14×159, the actual depth is 15.0 in.

Chapter 3 Selected Answers, cont'd.

13. What are the nominal and actual depths of a W10×100 wide-flange member? Compare these to the nominal and actual depths of a W10×49. (Hint: Use your AISC *Manual*.)

(Partial answer) from Manual Table 1-1, for the W10×100, the actual depth is 11.1 in.; for the W10×49, the actual depth is 10.0 in.

15. What are the actual depth, flange width, and flange thickness of a W27×129? (Hint: Use your AISC *Manual*.)

(Partial answer) from Manual Table 1-1, the flange width is 10.0 in., and the flange thickness is 1.10 in.

17. What are the actual depth, average flange thickness, and web thickness of a C12×25? (Hint: Use your AISC *Manual*.)

(Partial answer) from Table 1-5, the actual depth is 12.0 in.; the average flange thickness is 0.501 in.

19. What are the cross sectional area, leg dimensions and thickness of an L6×6×5/8? (Hint: Use your AISC *Manual*.)

From Manual Table 1-7, the cross sectional area is 7.13 in^2*; the leg dimensions are both 6 in., and thickness is 5/8 in.*

21. What are the cross sectional area and weight per linear foot of an L4×3×1/2 member? (Hint: Use your AISC *Manual*.)

From Manual Table 1-7, the cross sectional area is 3.25 in^2*, and the weight per linear foot is 11.1 lb/ft.*

23. What are the actual depth, flange width, flange thickness, and stem thickness of a WT13.5×42? Compare to the properties for a W27×84. (Hint: Use your AISC *Manual*.)

(Partial answer) from Manual Table 1-8, the actual depth is 13.4 in., stem thickness is 0.460 in. Compare to actual depth of a W27×84, from Manual Table 1-1, of 26.7 in., web thickness of 0.460 in.

25. What are the outside dimensions of a rectangular HSS10×6×1/2? What are the nominal and design wall thicknesses? (Hint: Use your AISC *Manual*.)

(Partial answer) from Manual Table 1-11, the design wall thickness is 0.465 in.

27. What is the outside diameter of a round HSS10.000×0.375? What are the nominal and design wall thicknesses? (Hint: Use your AISC *Manual*.)

(Partial answer) from Manual Table 1-13, the design wall thickness is 0.349 in.

29. What are the outside diameter and nominal and design wall thicknesses of a Pipe 10 xx-Strong? (Hint: Use your AISC *Manual*.)

(Partial answer) from Manual Table 1-14, this member has a design thickness of 0.930 in.

31. What is the difference between a rectangular bar section and a plate?

For the answer, see Section 3.4.3

Chapter 3 Selected Answers, cont'd.

33. What effects does the addition of carbon have on steel?

For the answer, see Section 3.5

35. What grade of steel is most commonly used today in the production of W-shapes, and what are its yield stress and tensile stress?

(Partial answer) from Section 3.6, A992, 50 ksi

37. What are the differences between an A500 Grade C rectangular HSS and an A1085 rectangular HSS?

(Partial answer), from Section 3.6 and Figure 3.10, they have the same specified minimum yield stress, but different tensile stress. A1085 also has a specific range for yield stress, from 50 to 70 ksi.

39. What grade of steel is typically used for high-strength bolts in construction?

For the answer, see Section 3.6.3

Chapter 4 Selected Answers

1. $A_g = 7.5\ in^2$; $A_n = 6.75\ in^2$

3. $A_g = 7.5\ in^2$; $A_n = 6.76\ in^2$

5. $A_g = 9.0\ in^2$; $A_n = 7.69\ in^2$

7. $A_g = 4.79\ in^2$; $A_n = 3.92\ in^2$

9. $A_g = 9.79\ in^2$; $A_n = 7.17\ in^2$

11. $A_g = 10.0\ in^2$; $A_n = 8.42\ in^2$

13. $A_g = 15.6\ in^2$; $A_n = 12.9\ in^2$

15. $A_g = 8.81\ in^2$; $A_n = 6.87\ in^2$

17. $w_n = 8.04$ in

19. $A_n = 3.97\ in^2$

21. $A_n = 5.97\ in^2$

23. $U = 0.85$; $A_e = 11.0\ in^2$

25. $U = 0.69$; $A_e = 6.90\ in^2$

27. $U = 0.838$; $A_e = 4.97\ in^2$

29. (a) $\phi P_n = 194$ kips; (b) $P_n/\Omega = 129$ kips

31. (a) $\phi P_n = 139$ kips; (b) $P_n/\Omega = 92.4$ kips

33. (a) $\phi P_n = 122$ kips; (b) $P_n/\Omega = 81.3$ kips

Chapter 4 Selected Answers, cont'd.

35. (a) ϕP_n = 375 kips; (b) P_n/Ω = 250 kips

37. (a) ϕP_n = 214 kips; (b) P_n/Ω = 143 kips

39. (a) ϕP_n = 177 kips; (b) P_n/Ω = 118 kips

41. (a) use L4×3×5/16; (b) use L4×4×3/8

43. (a) use WT6×26.5; (b) use WT6×25

45. (a) use W12×40; (b) use W12×40

47. (a) ϕR_n = 105 kips; (b) R_n/Ω = 69.9 kips

49. (a) ϕR_n = 527 kips; (b) R_n/Ω = 352 kips

51.1. (a) ϕP_n = 433 kips for controlling limit state of net section rupture;
(b) P_n/Ω = 289 kips for controlling limit state of net section rupture

51.2. (a) ϕP_n = 293 kips for controlling limit state of net section rupture;
(b) P_n/Ω = 196 kips for controlling limit state of net section rupture

53. (a) ϕP_n = 373 kips, controlling limit state is yielding;
(b) P_n/Ω = 172 kips, controlling limit state is yielding

55. (a) ϕP_n = 267 kips for controlling limit state of net section rupture;
(b) P_n/Ω = 178 kips for controlling limit state of net section rupture

Chapter 5 Selected Answers

1. P_{cr} = 303 kips; theoretical column will buckle

3. P_{cr} = 5760 kips; theoretical column will yield at 5450 kips

5. KL = 19.2 ft

7. KL = 12.3 ft

9. $(KL)_y$ = 6.53 ft

11. elastic buckling -- (a) ϕP_n = 663 kips; (b) P_n/Ω = 441 kips

13. inelastic buckling -- (a) ϕP_n = 145 kips; (b) P_n/Ω = 96.3 kips

15. elastic buckling -- (a) ϕP_n =1006 kips; (b) P_n/Ω = 669 kips

17. inelastic buckling -- (a) ϕP_n = 157 kips; (b) P_n/Ω =105 kips

19. inelastic buckling -- (a) ϕP_n = 674 kips; (b) P_n/Ω =449 kips

Chapter 5 Selected Answers, cont'd.

21. part (i) -- (a) ϕP_n = 339 kips; (b) P_n/Ω = 226 kips

part (ii) -- (a) ϕP_n = 428 kips; (b) P_n/Ω = 285 kips

23. L_{cx} = 203 in; L_{cy} = 250 in; L_{cy} (weak axis) controls column strength

25. Yes, W14×257 will carry load

(a) ϕP_n = 2900 kips > P_u = 2720 kips; (b) P_n/Ω = 1930 kips > P_a = 1900 kips

27. Yes, W8×58 will support load

(a) ϕP_n = 244 kips > P_u = 232 kips; (b) P_n/Ω = 163 kips = P_a = 160 kips

29. Yes, W16×77 will support load

(a) ϕP_n = 510 kips > P_u = 476 kips; (b) P_n/Ω = 340 kips > P_a = 330 kips

31. Yes, W24×146 will support load

(a) ϕP_n = 1330 kips > P_u = 1094 kips; (b) P_n/Ω = 883 kips > P_a = 745 kips

33. L_{cx} = 14 ft; L_{cy} = 10 ft; y-axis controls

35. For simplicity, brace every 5 ft

37. L_{cx} = 34.5 ft -- (a) ϕP_n = 610 kips; (b) P_n/Ω = 406 kips

39. L_{cx} = 26.3 ft -- (a) ϕP_n = 735 kips; (b) P_n/Ω = 489 kips

41. L_{cx} = 24.0 ft -- (a) ϕP_n = 766 kips; (b) P_n/Ω = 510 kips

43. L_{cx} = 36.0 ft -- (a) ϕP_n = 586 kips; (b) P_n/Ω = 390 kips

45. (a) select W12×50; (b) select W12×50

47. (a) select W10×60; (b) select W10×60

49. (a) select W6×15; (b) select W6×15

51. (a) select W12×65; (b) select W12×65

53. (a) select W14×74; (b) select W14×74

55. (a) select W14×74; (b) select W14×74

57. (a) use L4×4×3/8; (b) use L4×4×3/8

59. Yes W14×43 (Gr. 70) has a slender web and must be considered a slender element member

61. part (i) -- (a) ϕP_n = 382 kips; (b) P_n/Ω = 254 kips

part (ii) -- (a) ϕP_n = 247 kips; (b) P_n/Ω = 164 kips

63. (a) ϕP_n = 148 kips; (b) P_n/Ω = 98.2 kips

65. (a) ϕP_n = 14.8 kips; (b) P_n/Ω = 9.82 kips

Chapter 5 Selected Answers, cont'd.

67. (a) ϕP_n = 84.2 kips, compare to ϕP_n = 107 kips on AISC Table 4.9, which is not applicable for snug-tight bolts;

(b) P_n/Ω = 56.0 kips, compare to P_n/Ω = 71.3 kips on AISC Table 4.9, which is not applicable for snug-tight bolts

69. (a) ϕP_n = 21.4 kips; (b) P_n/Ω = 14.3 kips

71.

	(a) LRFD	(b) ASD
col. iv	use W8×31	use W8×31
col. v	use W10×60	use W12×65
col. vi	use W10×49	use W12×53

Chapter 6 Selected Answers

1. (partial answer) S = 753 in^3; Z = 853 in^3

3. (partial answer) S = 398 in^3; Z = 460 in^3

5. (partial answer) S = 56.5 in^3; Z = 65.2 in^3

7. (partial answer) Z = 241 in^3

9. (partial answer) $\bar{y}$ = 4.10 in. from top of section; S = 27.1in^3; y_p = 0.961 in. from top of section; Z = 49.2 in^3

11. (partial answer) $\bar{y}$ = 0.911 in. from top of section; S = 2.49 in^3; y_p = 0.406 in. from top of section; Z = 4.61 in^3

13. (partial answer) Z = 87.0 in^3

15. (partial answer) Z = 42.3 in^3

17. (partial answer) $\bar{y}$ = 4.00 in. from top of section; S = 8.10 in^3; y_p = 4.00 in. from top of section; Z = 9.57 in^3

19. (partial answer) $\bar{y}$ = 1.18 in. from bottom of section; S = 1.97 in^3; y_p = 0.470 in. from bottom of section; Z = 3.56 in^3

21. (partial answer) $\bar{y}$ = 0.840 in. from bottom of section; S = 0.574 in^3; y_p = 0.240 in. from bottom of section; Z = 1.04 in^3

23. (a) select W21×44; (b) select W21×44

25. (a) $Z_{req'd}$ = 373 in^3, use W30×116; (b) $Z_{req'd}$ = 376 in^3, use W30×116

27. (a) use W30×116; (b) use W33×118

29. (a) use W12×22; (b) use W14×22

Chapter 6 Selected Answers, cont'd.

31. use W27×84 for all cases

33. (a) use W16×67; (b) use W24×68

35. (a) select W18×35; (b) select W18×35

37. (a) locate lateral supports at fifth points, L_b = 7.20 ft; (b) locate lateral supports at fifth points, L_b = 7.20 ft

39. (a) use W18×55; (b) use W21×55

41. (a) use W18×35; (b) use W18×35

43. selected answers not provided

45. selected answers not provided

47. $b_f/2t_f$ = 6.47, compact flange

49. (a) ϕM_n = 1450 kip-ft; (b) M_n/Ω = 964 kip-ft

51. (a) ϕV_n = 306 kips; (b) V_n/Ω = 204 kips

53. (a) ϕV_n = 505 kips; (b) V_n/Ω = 337 kips

55. (a) ϕV_n = 740 kips; (b) V_n/Ω = 493 kips

57. (a) use W12×65; (b) use W16×67

59. (a) use W30×116; (b) use W33×118

61. (a) use W16×36; (b) use W16×40

63. Δ_{max} = 1.53 in.

65. (a) use W12×14; (b) use W12×26

67. use W16×31

69. (a) ϕM_n = 288 kip-ft; (b) M_n/Ω = 192 kip-ft

71. (a) ϕM_n = 780 kip-ft; (b) M_n/Ω = 519 kip-ft

73. (a) ϕM_n = 113 kip-in.; (b) M_n/Ω = 74.9 kip-in.

75. (a) ϕM_n = 113 kip-in.; (b) M_n/Ω = 74.9 kip-in.

77. (a) ϕM_n = 24.8 kip-ft; (b) M_n/Ω = 16.5 kip-ft

79. (a) ϕM_n = 84.9 kip-ft; (b) M_n/Ω = 56.5 kip-ft

81. (a) ϕM_n = 190 kip-ft; (b) M_n/Ω = 126 kip-ft

83. (a) use l_b = 3.0 in. (practical minimum)
(b) use l_b = 3.0 in. (practical minimum)

Chapter 6 Selected Answers, cont'd.

85. (a) use l_b = 3.0 in. (practical minimum)
(b) use l_b = 3.0 in. (practical minimum)

87. using SJI Tables

	(a) LRFD	(b) ASD
i	select 20K10	select 20K10
ii	select 22K11	select 22K11
iii	no K-series joist will satisfy strength requirement	no K-series joist will satisfy strength requirement

89.

	(a) LRFD	(b) ASD
1	select W21×62	select W24×68
2	select W18×35	select W18×35
3	select W18×35	select W16×40

Chapter 7 Selected Answers

1. (a) ϕM_n = 2490 kip-ft; (b) M_n/Ω = 1660 kip-ft

3. (a) ϕM_n = 2970 kip-ft; (b) M_n/Ω = 1980 kip-ft

5. (a) ϕM_n = 3010 kip-ft; (b) M_n/Ω = 2000 kip-ft

7. M_n = 463 kip-ft

9. M_n = 674 kip-ft

11. (a) ϕM_n = 2970 kip-ft; (b) M_n/Ω = 1980 kip-ft

13. (a) ϕM_n = 1080 kip-ft; (b) M_n/Ω = 716 kip-ft

15. (a) ϕM_n = 1970 kip-ft; (b) M_n/Ω = 1310 kip-ft

17. (a) ϕM_n = 1450 kip-ft; (b) M_n/Ω = 962 kip-ft

19. (a) ϕM_n = 2090 kip-ft; (b) M_n/Ω = 1390 kip-ft

21. (a) ϕV_n = 958 kips; (b) V_n/Ω = 638 kips

23. (a) ϕV_n = 362 kips; (b) V_n/Ω = 241 kips

Chapter 7 Selected Answers, cont'd.

25. (a) ϕV_n = 567 kips; (b) V_n/Ω = 377 kips

27. (a) ϕV_n = 356 kips; (b) V_n/Ω = 237 kips

29. In all but the end panel, (a) ϕV_n = 1410 kips; (b) V_n/Ω = 940 kips

31. (a) and (b) -- Yes, pair of 3 × 1/4 in. stiffener plates satisfy requirements

(No required shear strength values provided, therefore ρ_w (maximum shear ratio) could not be calculated, and the answer is the same for LRFD and ASD.)

33. One possible solution: 50 × 1/2 in. web plate, 30 × 2 in. flange plates, no stiffeners

35. One possible solution: 60 × 7/16 in. web plate, 28 × 2 in. flange plates, single 6 in. x 1/2 in. stiffeners at 4 ft o.c. for outer 20 ft at each end (all A572 Gr. 50)

37. One possible solution: 75 × 1/2 in. web plate, 36 × 2 in. flange plates, no stiffeners in middle third of span (50 ft), single 7 in. x 5/8 in. stiffeners at 10 ft o.c. in outer thirds of span (all A572 Gr. 50)

Chapter 8 Selected Answers

1. (a) Eq. (H1-1a) gives 0.795 < 1.0, so yes, W14×90 is adequate

(b) Eq. (H1-1a) gives 0.801 < 1.0, so yes, W14×90 is adequate

3. (a) Eq. (H1-1a) gives 0.927 < 1.0, so yes, W12×190 is adequate

(b) Eq. (H1-1a) gives 0.971 < 1.0, so yes, W12×190 is adequate

5. (a) M_D = 221 kip-ft, M_L = 663 kip-ft; (b) M_D = 220 kip-ft, M_L = 660 kip-ft

7. (a) Eq. (H1-1a) gives 0.806 < 1.0, so yes, W14×90 is adequate

(b) Eq. (H1-1a) gives 0.812 < 1.0, so yes, W14×90 is adequate

9. (a) Eq. (H1-1a) gives 0.956 < 1.0, so yes, W12×190 is adequate

(b) Eq. (H1-1a) gives 1.00, so yes, W12×190 is adequate

11. (a) Eq. (H1-1a) gives 1.01 > 1.0, so no, W14×68 is not adequate

(b) Eq. (H1-1a) gives 1.01 > 1.0, so no, W14×68 is not adequate

13. (a) Eq. (H1-1a) gives 0.968 < 1.0, so yes, W14×109 is adequate

(b) Eq. (H1-1a) gives 0.977 < 1.0, so yes, W14×109 is adequate

15. (a) Eq. (H1-1a) gives 0.813 < 1.0, so yes, W14×61 is adequate

(b) Eq. (H1-1a) gives 0.821 < 1.0, so yes, W14×61 is adequate

Chapter 8 Selected Answers, cont'd.

17. (Partial answers) For load combinations with wind:

(a) Eq. (H1-1b) gives 0.467 < 1.0, so yes, column in upper story is adequate

Eq. (H1-1b) gives 0.318 < 1.0, so yes, column in lower story is adequate

(b) Eq. (H1-1b) gives 0.691 < 1.0, so yes, column in upper story is adequate

Eq. (H1-1a) gives 0.524 < 1.0, so yes, column in lower story is adequate

19. (a) Eq. (H1-1b) gives 0.525 < 1.0, so yes, left column is adequate

Eq. (H1-1a) gives 0.479 < 1.0, so yes, interior column is adequate

By inspection, right column is also adequate

(b) Eq. (H1-1b) gives 0.541 < 1.0, so yes, left column is adequate

Eq. (H1-1a) gives 0.498 < 1.0, so yes, interior column is adequate

By inspection, right column is also adequate

21. Eq. (H1-1a) gives 0.952 < 1.0, so yes, W14×82 is adequate

23. Eq. (H1-1a) gives 1.03 > 1.0, so no, W10×54 is not adequate

25a. Use W14×159; eq. (H1-1a) gives 0.969 < 1.0

25b. Use W14×159; eq. (H1-1a) gives 0.964 < 1.0

27a. Use W14×193; eq. (H1-1a) gives 0.908 < 1.0

27b. Use W14×193; eq. (H1-1a) gives 0.909 < 1.0

29. (a) Yes, given structure is adequate; (b) Yes, given structure is adequate

31. (a) Use 5/8 in. rod, A = 0.307 in^2; (b) Use 3/4 in. rod, A = 0.442 in^2 (11/16 in. would work, but not a standard diameter)

33. (a) P_{br} = 5.40 kips, β_{br} = 360 kip/ft; (b) P_{br} = 3.72 kips, β_{br} = 372 kip/ft

35. (a) Eq. (H1-1b) gives 0.799 < 1.0, so yes, W18×86 is adequate

(b) Eq. (H1-1b) gives 0.800 < 1.0, so yes, W18×86 is adequate

Chapter 9 Selected Answers

1. (a) ϕM_n = 376 kip-ft, PNA in concrete; (b) M_n/Ω = 250 kip-ft, PNA in concrete

3. (a) ϕM_n = 885 kip-ft, PNA in concrete; (b) M_n/Ω = 589 kip-ft, PNA in concrete

5. (a) ϕM_n = 2020 kip-ft, PNA in concrete; (b) M_n/Ω = 1340 kip-ft, PNA in concrete

7. (a) ϕM_n = 825 kip-ft, PNA in flange; (b) M_n/Ω = 549 kip-ft, PNA in flange

9. (a) ϕM_n = 632 kip-ft, PNA in flange; (b) M_n/Ω = 420 kip-ft, PNA in flange

Chapter 9 Selected Answers, cont'd.

11. (a) ϕM_n = 493 kip-ft, PNA in flange; (b) M_n/Ω = 328 kip-ft, PNA in flange

13. (a) ϕM_n = 328 kip-ft, PNA in flange; (b) M_n/Ω = 218 kip-ft, PNA in flange

15. (a) ϕM_n = 803 kip-ft, PNA in flange; (b) M_n/Ω = 534 kip-ft, PNA in flange

17. (a) ϕM_n = 402 kip-ft, PNA in web; (b) M_n/Ω = 268 kip-ft, PNA in web

19. (a) ϕM_n = 8250 kip-ft, PNA in flange; (b) M_n/Ω = 5490 kip-ft, PNA in flange

21. (a) ϕM_n = 99.0 kip-ft, PNA in flange; (b) M_n/Ω = 65.9 kip-ft, PNA in flange

23. (a) ϕM_n = 598 kip-ft, PNA in flange; (b) M_n/Ω = 398 kip-ft, PNA in flange
and plot M_n for V'_q = 250 kips, 400 kips, and 734 kips

25. (a) Use W14×22 with 28 – 3/4 in. studs; (b) Use W14×22 with 32 – 3/4 in. studs

27. (a) Use W16×40 with 40 – 3/4 in. studs; (b) Use W16×40 with 56 – 3/4 in. studs

29. (a) W18×35 with 26 shear studs, W16×36 with 30 shear studs; almost the same lb/ft but W16 requires more shear studs

(b) W18×35 with 32 shear studs, W16×36 with 36 shear studs; almost the same lb/ft but W16 requires more shear studs

31. (a) ϕM_n = 392 kip-ft, 40 (20 pairs) – 3/4 in. studs symmetrically placed wrt beam midspan, starting at the supports; (b) M_n/Ω = 260 kip-ft, 40 (20 pairs) – 3/4 in. studs symmetrically placed wrt beam midspan, starting at the supports

33. (a) ϕM_n = 436 kip-ft, 44 (22 pairs) – 3/4 in. studs symmetrically placed wrt beam midspan, starting at the supports; (b) M_n/Ω = 290 kip-ft, 44 (22 pairs) – 3/4 in. studs symmetrically placed wrt beam midspan, starting at the supports

35. (a) ϕM_n = 632 kip-ft; (b) M_n/Ω = 420 kip-ft

37. (a) Use W18×35 with 3/4 in. studs distributed 25-4-25 over each third of the span; (b) Use W18×35 with 3/4 in. studs distributed 29-4-29 over each third of the span

39. (a) Use W18×35 with 21 - 3/4 in. studs each side of the concentrated load; (b) Use W18×35 with 24 - 3/4 in. studs each side of the concentrated load

41. Δ_L = 0.549 in.

43. (a) Δ_L = 0.618 in.; (b) Δ_L = 0.595 in.

45. (Partial answer) (a) Use W14×38; (b) Use W14×38

47. (a) ϕP_n = 1540 kips; (b) P_n/Ω = 1030 kips

49. (a) ϕP_n = 3420 kips; (b) P_n/Ω = 2280 kips

51. (a) ϕP_n = 2380 kips; (b) P_n/Ω = 1580 kips

53. (a) ϕP_n = 408 kips; (b) P_n/Ω = 272 kips

Chapter 9 Selected Answers, cont'd.

55. (a) ϕP_n = 900 kips; (b) P_n/Ω = 600 kips

57. Yes, HSS 10×5×3/8 will carry load and moment; (a) ϕP_n = 323 kips, ϕM_n = 140 kip-ft; (b) P_n/Ω = 216 kips, M_n/Ω = 92.8 kip-ft

Chapter 10 Selected Answers

1. (Partial answer) Nominal shear strength for A325-N bolts

Bolt diameter (in)	5/8	3/4	7/8	1
Nominal bolt area (in^2)	0.307	0.442	0.601	0.785
Nominal shear strength, R_n (kips)	16.6	23.9		

3. (Partial answer) Nominal shear strength for A490-N bolts

Bolt diameter (in)	5/8	3/4	7/8	1
Nominal bolt area (in^2)	0.307	0.442	0.601	0.785
Nominal shear strength, R_n (kips)	20.9	30.1		

5. (Partial answer) Design shear strength (LRFD)

		Bolt diameter (in)	5/8	3/4	7/8	1
		Nominal bolt area (in^2)	0.307	0.442	0.601	0.785
Grade	N or X	F_{nv} (ksi)	Design shear strength, ϕR_n (kips)			
A325	N	54	12.4	17.9	24.3	31.8
A325	X					
A490	N	68	15.7	22.5		
A490	X					

7. (Partial answer) Design shear strength (LRFD)

		Bolt diameter (in)	5/8	3/4	7/8	1
		Nominal bolt area (in^2)	0.307	0.442	0.601	0.785
Grade	N or X	F_{nv} (ksi)	Design shear strength, ϕR_n (kips)			
F1852	N	54	12.4	17.9	24.3	31.8
F1852	X					
F2280	N	68	15.7	22.5		
F2280	X					

Chapter 10 Selected Answers, cont'd.

9. (a) ϕR_n = 107 kips; (b) R_n/Ω = 71.5 kips

11. (a) ϕR_n = 130 kips; (b) R_n/Ω = 87.0 kips

13. (a) ϕR_n = 152 kips; (b) R_n/Ω = 101 kips

15. (a) ϕR_n = 122 kips; (b) R_n/Ω = 81.0 kips

17. (a) ϕR_n = 136 kips; (b) R_n/Ω = 90.5 kips

19. (a) ϕR_n = 215 kips; (b) R_n/Ω = 143 kips

21. selected answers not provided

23. selected answers not provided

25. (a) ϕR_n = 134 kips; (b) R_n/Ω = 89.8 kips

27. (a) ϕR_n = 95.8 kips; (b) R_n/Ω = 63.8 kips

29. (a) ϕR_n = 84.2 kips; (b) R_n/Ω = 56.1 kips

31. (a) ϕR_n = 146 kips; (b) R_n/Ω = 97.2 kips

33. (a) ϕR_n = 180 kips; (b) R_n/Ω = 120 kips

35. (a) ϕR_n = 305 kips; (b) R_n/Ω = 203 kips

37. (a) ϕR_n = 267 kips; (b) R_n/Ω = 178 kips

39. (a) ϕR_n = 250 kips; (b) R_n/Ω = 167 kips

41. (a) ϕR_n = 133 kips; (b) R_n/Ω = 88.5 kips

43. selected answers not provided

45. (a) ϕR_n = 146 kips; (b) R_n/Ω = 97.5 kips

47. (a) ϕR_n = 49.8 kips; (b) R_n/Ω = 33.2 kips

49. (a) ϕR_n = 180 kips; (b) R_n/Ω = 120 kips

51. (a) ϕR_n = 475 kips; (b) R_n/Ω = 316 kips

53. (a) ϕR_n = 482 kips; (b) R_n/Ω = 321 kips

Chapter 11 Selected Answers

1. 3 bolts, pair of 4×3-1/2×5/16×8.5 in. angles, edge distances of 1.25 in.

3. 5 bolts, pair of 4×3-1/2×5/16×14.5 in. angles, edge distances of 1.25 in.

5. 4 bolts, pair of 3-1/2×3-1/2×3/8×11.5 in. angles, edge distances of 1.25 in.

7. 6 bolts, pair of 3-1/2×3-1/2×5/16×17.5 in. angles, edge distances of 1.25 in.

9. 4 bolts, pair of 3-1/2×3-1/2×5/16×11.5 in. angles, edge distances of 1.25 in.

11. 4 bolts, pair of 3-1/2×3-1/2×5/16×11.5 in. angles, edge distances of 1.25 in.

13. 5 bolts, pair of 3-1/2×3-1/2×5/16×14.5 in. angles, edge distances of 1.25 in; increase the distance of the top bolt to the coped edge to 1-5/8 in.

15. 3 bolts, pair of 3-1/2×3-1/2×5/16×8.5 in. angles, edge distances of 1.25 in.

17. 4 bolts, pair of 3-1/2×3-1/2×3/8×11.5 in. angles, edge distances of 1.25 in.

19. 3 bolts, pair of 4×3-1/2×5/16×8.5 in. angles, edge distances of 1.25 in., 3/16 in. fillet welds

21. 5 bolts, pair of 4×3-1/2×5/16×14.5 in. angles, edge distances of 1.25 in., 3/16 in. fillet welds

23. 3 bolts, pair of 3-1/2×3-1/2×5/16×8.5 in. angles, edge distances of 1.25 in., 3/16 in. fillet welds

25. 3 bolts, pair of 3-1/2×3-1/2×5/16×8.5 in. angles, edge distances of 1.25 in., 3/16 in. fillet welds

27. 4 bolts in a 3-1/2×3-1/2×3/8×11.5 in. angle, edge distances of 1.25 in.

29. 3 bolts in a 3-1/2×3-1/2×3/8×8.5 in. angle, edge distances of 1.25 in., 1/4 in. fillet welds

31. 3-1/2×3-1/2×3/8×10 in. angle with 1/4 in. welds

33. 4-1/2×11-1/2×3/8 plate with 4 bolts and 1/4 in. fillet welds

35. 4-1/2×11-1/2×5/16 plate with 4 bolts and 1/4 in. fillet welds

37. 4-1/2×11-1/2×3/8 plate with 4 bolts and 1/4 in. fillet welds

39. L6×6×1, 8.0 in. long with 3/16 in. fillet welds to the column

41. L4×4×5/8, 8.0 in. long with 7/16 in. fillet welds to the column

43. (a) ϕR_n = 104 kips; (b) R_n/Ω = 69.6 kips

45. Use a 2-bolt connection with a 4 x 1/2 in. plate and ¼ in. fillet weld each side to the supporting member

Chapter 11 Selected Answers, cont'd.

47.1. The bolts are adequate by either LRFD or ASD.

47.2. The bolts are not adequate by either LRFD or ASD.

49. Prying action must be considered for LRFD and ASD.

51. (a) ϕR_n = 36.3 kips; (b) R_n/Ω = 24.2 kips

53. (a) and (b) Use a 5/8×7-1/2×10 in. A572 Gr. 50 plate

55. (a) and (b) Use a 1-1/4×15×1 ft – 2 in. A572 Gr. 50 plate

Chapter 12 Selected Answers

1. The flange plates of this connection are PL1/2×6-1/2×0'-10" with six 3/4 in. A325-N bolts and a pair of 1/4 fillet welds to the column flange.

The web plate is PL3/8×3-1/2×0'-9" with three 3/4 in. A325-N bolts and a pair of 3/16 in. fillet welds to the column flange.

2. None of the limit states checked require stiffeners or doubler plates. Therefore no stiffener or doubler plate design is required.

3. The top flange plate is PL1-1/4×9-1/2×2'-2-1/4". The bottom flange plate is PL7/8×11-1/2×2'-2-1/4". Both plates are connected to the beam flange with 3/8 in. fillet welds for a length of 24-1/4 in. The top plate is connected to the column flange with a pair of 5/8 in. fillet welds and the bottom plate with a pair of 9/16 in. fillet welds.

The web plate is PL3/8×3-1/2×1'-0" with four 3/4 in. A325-N bolts and a pair of 3/16 in. fillet welds to the column flange.

4. (Partial answer) A doubler plate is required for the web panel zone shear.

5. The flange plates of this connection are PL5/8×7-1/2×1'-1" with eight 7/8 in. A325-N bolts and a pair of 3/8 in. fillet welds to the column flange.

The web plate is PL3/8×3-1/2×0'-9" with three 7/8 in. A325-N bolts and a pair of 3/16 in. welds to the column flange.

15. CJP welds for the beam flanges; the web plate is PL3/8×3-1/2×0'-9" with three 3/4 in. A325-N bolts and a pair of 3/16 in. fillet welds to the column flange.

16. None of the limit states checked require stiffeners or doubler plates. Therefore no stiffener or doubler plate design is required.

17. CJP welds for the beam flanges; the web plate is PL3/8×3-1/2×1'-0" with four 3/4 in. A325-N bolts and a pair of 3/16 in. fillet welds to the column flange.

18. (Partial answer) A doubler plate is required for the web panel zone shear.

Chapter 13 Selected Answers

1. What is the major difference between the analysis and design of a structure for wind and gravity loads and the analysis and design of a structure for seismic loads?

For the answer, see Section 13.1

3. How are the R, C_d, and Ω_o factors determined for a particular analysis?

For the answer, see Section 13.1

5. How is the R factor used to influence the seismic forces a structure is designed to resist?

For the answer, see Section 13.1

7. Explain the capacity design method in relation to material yield strength.

For the answer, see Section 13.2

9. What are the three primary types of moment frames considered in the Seismic Provisions? What are the respective values for R, C_d, and Ω_o for each of these systems?

For the answer, see Section 13.3

11. Which type of moment frame is required to meet an interstory drift angle of 0.02 radians?

For the answer, see Section 13.3

13. List the three primary types of braced frame systems addressed in the Seismic Provisions and their corresponding values for R, C_d, and Ω_o.

For the answer, see Section 13.4

15. List some examples of CBF configurations.

For the answer, see Section 13.4

17. Where are the fuse elements located for eccentrically braced frames?

For the answer, see Section 13.4

19. Describe how SPSW resist seismic forces.

For the answer, see Section 13.5

21. How does the size of connections in seismic force resisting systems differ from that of connections designed for gravity and wind systems? Why?

For the answer, see Section 13.6

23. For seismic design, where should column splices be located and why?

For the answer, see Section 13.6