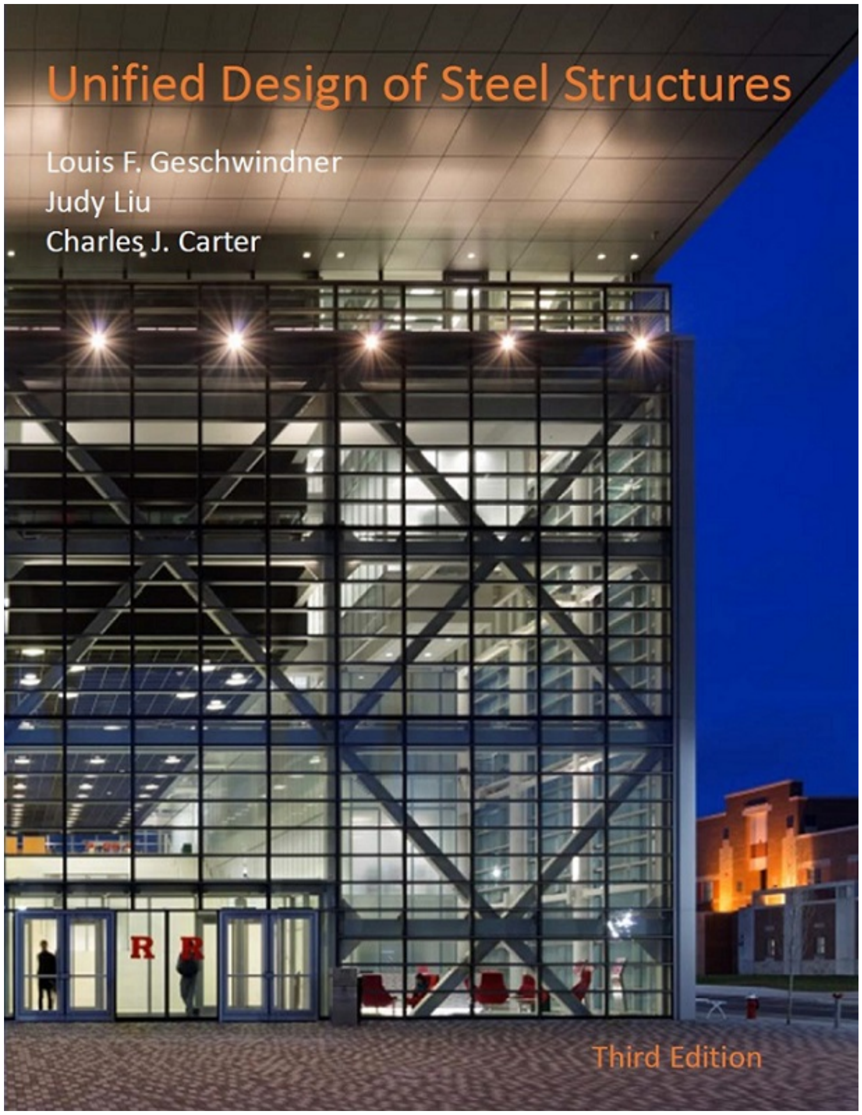

Unified Design of Steel Structures, 3rd Edition

Selected Homework Problem Answers

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
- **d.** design of members for combined forces and tension
- **e.** requirements for design of structures to ensure stability

For the answer, see Section 1.2

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

For the answer, see Section 1.5

7. Provide an example of each of the following types of construction. To the extent possible, identify specific buildings in your own locale.

- **a.** Bearing wall
- **b.** Beam-and-column
- **c.** Long-span
- **d.** High-rise
- **e.** Gable-frame

For the answer, see Section 1.7

9. List and describe two types of lateral load resisting systems commonly used in beam-and-column construction.

For the answer, see Section 1.7

11. Provide a simple definition of structural design.

For the answer, see Section 1.8

13. Give a description of both the LRFD and ASD design approaches. What is the fundamental difference between the methods?

For the answer, see Sections 1.9 and 1.10

15. Identify three sources of variation in the strength of a structure and its components.

For the answer, see Section 1.12

17. 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. What analysis method allows the designer to visualize the load on a particular structural element without performing an actual equilibrium calculation?

For the answer, see Section 2.3

7. In determining the snow load on a structure, what value that can be obtained from the applicable building code is multiplied by a series of factors to obtain the roof snow load?

For the answer, see Section 2.3.3

9. Name four factors that must be taken into account in converting wind speed data referenced by the building code into wind pressure on a given building.

For the answer, see Section 2.3.4

11. 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

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

For the answer, see Section 2.4

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

For the answer, see Table 2.2 or ASCE 7 Table 4-1

17. Determine the nominal uniformly distributed self-weight of a 6 in. thick reinforced concrete slab.

Use Table 2.1 to obtain 75 psf

19. If the framing plan shown in Figure P2.19 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.

Chapter 2 Selected Answers, Problem 19, cont'd.

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 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

9. What are the nominal and actual depths of a W16×100 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 17.0 in.

11. What are the nominal and actual depths of a W14×730 wide-flange member? Compare these to the nominal and actual depths of a W14×145. (Hint: Use your AISC *Manual*.)

(Partial answer) from Manual Table 1-1, for the W14×730, the actual depth is 22.4 in.; for the W14×145, the actual depth is 14.8 in.

13. What are the nominal and actual depths of a W10×112 wide-flange member? Compare these to the nominal and actual depths of a W10×12. (Hint: Use your AISC *Manual*.)

(Partial answer) from Manual Table 1-1, for the W10×112, the actual depth is 11.4 in.; for the W10×12, the actual depth is 9.87 in.

15. What are the actual depth, flange width, and flange thickness of a W27×84? (Hint: Use your AISC *Manual*.)

(Partial answer) from Manual Table 1-1, the flange width is 10.0 in., and the flange thickness is 0.64 in.

17. What are the actual depth, average flange thickness, and web thickness of a C12×30? (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 L8×8×5/8? (Hint: Use your AISC *Manual*.)

From Manual Table 1-7, the cross sectional area is 9.69 in^2; the leg dimensions are both 8 in., and thickness is 5/8 in.

Chapter 3 Selected Answers, cont'd.

21. What are the cross sectional area and weight per linear foot of an L5×3×1/2 member? (Hint: Use your AISC *Manual*.)

From Manual Table 1-7, the cross sectional area is 3.75 in^2, and the weight per linear foot is 12.8 lb/ft.

23. What are the actual depth, flange width, flange thickness, and stem thickness of a WT10.5×22? Compare to the properties for a W21×44. (Hint: Use your AISC *Manual*.)

(Partial answer) from Manual Table 1-8, the actual depth is 10.3 in., stem thickness is 0.350 in. Compare to actual depth of a W21×44, from Manual Table 1-1, of 20.7 in., web thickness of 0.350 in.

25. What are the outside dimensions of a rectangular HSS8×6×1/2? 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 HSS10.750×0.375? 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 4 xx-Strong? (Hint: Use your AISC *Manual*.)

(Partial answer) from Manual Table 1-14, this member has a design thickness of 0.628 in.

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

For the answer, see Section 3.4.3

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. $A_g = 6$ in^2; $A_n = 5.25$ in^2

3. $A_g = 3.75$ in^2; $A_n = 3.01$ in^2

5. $A_g = 7.5$ in^2; $A_n = 6.41$ in^2

7. $A_g = 3.75$ in^2; $A_n = 2.88$ in^2

9. $A_g = 6.49$ in^2; $A_n = 4.80$ in^2

11. $A_g = 9.11$ in^2; $A_n = 7.50$ in^2

13. $A_g = 7.34$ in^2; $A_n = 5.72$ in^2

15. $A_g = 11.8$ in^2; $A_n = 9.75$ in^2

17. $w_n = 8.04$ in

19. $A_n = 4.13$ in^2

21. $A_n = 5.98$ in^2

23. $U = 0.85$; $A_e = 10.8$ in^2

25. $U = 0.69$; $A_e = 6.9$ in^2

27. $U = 0.838$; $A_e = 4.96$ in^2

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

31. (a) $\phi P_n = 109$ kips; (b) $P_n/\Omega = 72.5$ kips

33. (a) $\phi P_n = 122$ kips; (b) $P_n/\Omega = 81.0$ kips

35. (a) $\phi P_n = 214$ kips; (b) $P_n/\Omega = 143$ kips

37. (a) $\phi P_n = 140$ kips; (b) $P_n/\Omega = 93.5$ kips

39. (a) use L4×4×7/16; (b) use L5×3-1/2×1/2

41. (a) use WT6×26.5; (b) use WT6×26.5

43. (a) use W14×68; (b) use W14×68

45. (a) $\phi R_n = 225$ kips; (b) $R_n/\Omega = 150$ kips

47. (a) $\phi R_n = 310$ kips; (b) $R_n/\Omega = 206$ kips

49. (a) $\phi R_n = 256$ kips; (b) $R_n/\Omega = 171$ kips

Chapter 5 Selected Answers

1. P_{cr} = 373 kips; theoretical column will buckle

3. P_{cr} = 10,250 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 = 660 kips; (b) P_n/Ω = 439 kips

13. inelastic buckling -- (a) ϕP_n = 145 kips; (b) P_n/Ω = 96.4 kips

15. elastic buckling -- (a) ϕP_n =1010 kips; (b) P_n/Ω = 672 kips

17. inelastic buckling -- (a) ϕP_n = 158 kips; (b) P_n/Ω =105 kips

19. inelastic buckling -- (a) ϕP_n = 643 kips; (b) P_n/Ω =428 kips

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, W8×67 will support load

(a) ϕP_n = 241 kips > P_u = 232 kips; (b) P_n/Ω = 160 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, W24×131 will support load

(a) ϕP_n = 1180 kips > P_u = 1090 kips; (b) P_n/Ω = 784 kips > P_a = 745 kips

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

35. For simplicity, brace every 5.00 ft

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

39. weak axis controls, assuming braced frame out of plane

(a) strong axis ϕP_n = 745 kips, L_{cx} = 25.5 ft; weak axis ϕP_n = 735 kips, L_{cy} = 15 ft

(b) strong axis P_n/Ω = 495 kips, L_{cx} = 25.5 ft; weak axis P_n/Ω = 489 kips, L_{cy} = 15 ft

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

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

Chapter 5 Selected Answers, cont'd.

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

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

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

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

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

55. part (i) -- (a) ϕP_n = 232 kips; (b) P_n/Ω = 154 kips

part (ii) -- (a) ϕP_n = 104 kips; (b) P_n/Ω = 69.5 kips

57. (a) ϕP_n = 130 kips; (b) P_n/Ω = 86.2 kips

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

61. (a) ϕP_n = 84.2 kips; (b) P_n/Ω = 56.0 kips

63.

	(a) LRFD	(b) ASD
col. iv	select W8×28	select W8×31
col. v	select W10×49	select W10×54
col. vi	select W8×35	select W8×40

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 = 25.8 in^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

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. solution not provided

45. solution not provided

47. (a) ϕV_n = 375 kips; (b) V_n/Ω = 250 kips

49. (a) use W24×146; (b) use W24×146

51. (a) use W16×36; (b) use W16×40

Chapter 6 Selected Answers, cont'd.

53. (a) use W18×35; (b) use W14×43

55. (a) use W36×135; (b) use W36×135

57. (a) ϕM_n = 6.45 kip-ft; (b) M_n/Ω = 4.29 kip-ft

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

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

63. using 2013 Vulcraft Steel Joists & Joist Girders catalog, pp. 47,52

	(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

65.

	(a) LRFD	(b) ASD
i	select W14×26	select W16×26
ii	select W24×68	select W24×76
iii	select W14×26	select W16×26

Chapter 7 Selected Answers

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

3. (a) ϕM_n = 2290 kip-ft; (b) M_n/Ω = 1520 kip-ft

5. M_n = 356 kip-ft

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

9. (a) ϕM_n = 1080 kip-ft; (b) M_n/Ω = 719 kip-ft

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

13. (a) ϕV_n = 954 kips; (b) V_n/Ω = 635 kips

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

17. (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.)

19. One possible solution: 50 × 9/16 in. web plate, 32 × 1-3/4 in. flange plates, no stiffeners

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 0.922 < 1.0, so yes, W12×190 is adequate

(b) Eq. (H1-1a) gives 0.959 < 1.0, so yes, W12×190 is adequate

5. (a) M_D = 583 kip-ft, M_L = 1750 kip-ft; (b) M_D = 582 kip-ft, M_L = 1750 kip-ft

7. (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

9. (a) Eq. (H1-1a) gives 0.922 < 1.0, so yes, W12×190 is adequate

(b) Eq. (H1-1a) gives 0.959 < 1.0, so yes, W12×190 is adequate

11. (a) Eq. (H1-1a) gives 0.999 < 1.0, so yes, W14×68 is adequate

(b) Eq. (H1-1a) gives 0.997 < 1.0, so yes, W14×68 is adequate

13. (a) Eq. (H1-1a) gives 0.878 < 1.0, so yes, W14×120 is adequate

(b) Eq. (H1-1a) gives 0.885 < 1.0, so yes, W14×120 is adequate

15. (a) Eq. (H1-1a) gives 1.15 > 1.0, so no, W14×48 is not adequate

(b) Eq. (H1-1a) gives 1.16 > 1.0, so no, W14×48 is not adequate

17. (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 1.05 > 1.0, so no, W14×74 is not adequate

23. Eq. (H1-1a) gives 0.900 < 1.0, so yes, W10×60 is adequate

25a. Use W14×176; eq. (H1-1a) gives 0.925 < 1.0

25b. Use W14×176; eq. (H1-1a) gives 0.919 < 1.0

Chapter 8 Selected Answers, cont'd.

27a. Use W14×176; eq. (H1-1a) gives 0.971 < 1.0

27b. Use W14×176; eq. (H1-1a) gives 0.972 < 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} = 5.42 kips, β_{br} = 362 kip/ft; (b) P_{br} = 3.74 kips, β_{br} = 374 kip/ft

35. (a) Eq. (H1-1b) gives 0.786 < 1.0, so yes, W18×86 is adequate

(b) Eq. (H1-1b) gives 0.788 < 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 = 456 kip-ft, PNA in flange; (b) M_n/Ω = 304 kip-ft, PNA in flange

7. (a) ϕM_n = 10,100 kip-ft, PNA in concrete; (b) M_n/Ω = 6710 kip-ft, PNA in concrete

9. (a) ϕM_n = 122 kip-ft, PNA in concrete; (b) M_n/Ω = 81.4 kip-ft, PNA in concrete

11. (a) ϕM_n = 551 kip-ft, PNA in flange; (b) M_n/Ω = 366 kip-ft, PNA in flange

13. (a) ϕM_n = 788 kip-ft, PNA in web; (b) M_n/Ω = 524 kip-ft, PNA in web

15. (a) ϕM_n = 626 kip-ft, PNA in flange; (b) M_n/Ω = 416 kip-ft, PNA in flange

17. (a) ϕM_n = 1850 kip-ft, PNA in flange; (b) M_n/Ω = 1230 kip-ft, PNA in flange

19. (a) ϕM_n = 447 kip-ft, PNA in flange; (b) M_n/Ω = 298 kip-ft, PNA in flange

And plot M_n versus V'_q for Problems 5, 14, and 19

21. (a) Use W12×14 with 16 – 3/4 in. studs; (b) Use W12×14 with 20 – 3/4 in. studs

23. (a) Use W16×26 with 10 – 3/4 in. studs; (b) Use W16×26 with 10 – 3/4 in. studs

25. (a) W14×26 with 32 shear studs, W16×26 with 28 shear studs; same lb/ft but W14 requires more shear studs

(b) W14×26 with 36 shear studs, W16×26 with 32 shear studs; same lb/ft but W14 requires more shear studs

27. (a) ϕM_n =507 kip-ft; (b) M_n/Ω = 337 kip-ft

29. (a) Use W12×14 with 18 – 3/4 in. studs; (b) Use W12×14 with 20 – 3/4 in. studs

31. Δ_L = 0.418 in.

33. (a) Δ_L = 0.436 in.; (b) Δ_L = 0.408 in.

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

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

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

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	68	15.7	22.5		
A490	N					
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	68	15.7	22.5		
F2280	N					
F2280	X					

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 = 170 kips; (b) R_n/Ω = 113 kips

Chapter 10 Selected Answers, cont'd.

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

17. (a) ϕR_n = 260 kips; (b) R_n/Ω = 174 kips

19. solution not provided

21. solution not provided

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

25. (a) ϕR_n = 53.7 kips; (b) R_n/Ω = 35.7 kips

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

29. (a) ϕR_n = 167 kips; (b) R_n/Ω = 112 kips

31. (a) ϕR_n = 89.2 kips; (b) R_n/Ω = 59.5 kips

33. (a) ϕR_n = 218 kips; (b) R_n/Ω = 145 kips

35. (a) ϕR_n = 109 kips; (b) R_n/Ω = 72.5 kips

37. (a) ϕR_n = 130 kips; (b) R_n/Ω = 86.5 kips

39. (a) ϕR_n = 214 kips; (b) R_n/Ω = 142 kips

41. (a) ϕR_n = 234 kips; (b) R_n/Ω = 156 kips

43. (a) ϕR_n = 297 kips; (b) R_n/Ω = 198 kips

45. (a) ϕR_n = 376 kips; (b) R_n/Ω = 251 kips

Chapter 11 Selected Answers

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

3. 5 bolts, pair of 3-1/2×3-1/2×5/16×14.5 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 3-1/2×3-1/2×5/16×8.5 in. angles, edge distances of 1.25 in., 3/16 in. fillet welds

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

23. 4 bolts, pair of 3-1/2×3-1/2×5/16×11.5 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. 3-1/2×3-1/2×3/8×10 in. angle with 1/4 in. welds

31. 4-1/2×17-1/2×5/16 plate with 6 bolts and 1/4 in. fillet welds

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

35. L4×4×5/8 and 3/8 in. welds

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

39. Use a 1-1/4×6×1’-0” plate

41. Use a 1-1/2×24×2’-0” 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 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 PL1-1/2×9-1/2×2’-2-1/4”. The bottom flange plate is PL1-1/4×11-1/2×2’-2-1/4”. Both plates are connected to the beam flange with 3/8 in. fillet welds for a length of 22-1/4 in. and a pair of 9/16 fillet welds to the column flange.

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 fillet welds to the column flange.

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

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 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 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 has a ductility requirement for connections of 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