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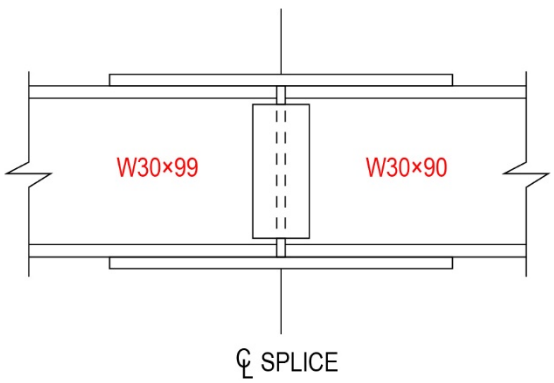
Revisions are shown in red.

Question 103, Vertical Forces, p. 14
The options should read as follows:

(A) 1.7
(B) 2.5
(C) 3.9
(D) 5.0

Question 123, Vertical Forces, p. 31
The options and graphic should read as follows:

(A) 764
(B) 895
(C) 957
(D) 1,020



Question 124, Vertical Forces, p. 32
The assumptions should read as follows:

The header is fully braced. Lateral-torsional buckling and distortional buckling need **not** be considered. The track sections stiffen the flanges of the 800S200 sections.

Solutions Table, Vertical Forces, p. 70
123: The correct answer is A.

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Solution 103, Vertical Forces, p. 71
The solution should read as follows:

ASCE 7, Figure 7.6-1 Formula.

$$\frac{h_d}{\sqrt{I_s}} = \left(0.43\sqrt[3]{l_u}\,\sqrt[4]{p_g + 10}\right) - 1.5$$

$p_g = 30,\qquad l_u = 100\text{ ft}\qquad$ (given)

Risk Category IV $\therefore\ I_s = 1.20$ — ASCE 7 Table 1.5-2

$$h_d = \left[\left(0.43\sqrt[3]{100}\,\sqrt[4]{30 + 10}\right) - 1.5\right]\left(\sqrt{1.20}\right) = 3.86\text{ ft}$$

Solution 123, Vertical Forces, p. 83
The solution should read as follows:

Strength I, M_u = 1.25(25.8 ft-kips) + 1.75(492.7 ft-kips) — AASHTO Table 3.4.1-1
= 894.5 ft-kips

THE CORRECT ANSWER IS: (B)

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Solution 124, Vertical Forces, p. 83
The solution should read as follows:

ASD option:

$$M_{n\ell} = S_e F_n \qquad \text{AISI Eq. F3.1-1}$$

Since header is fully braced, $F_n = F_y$

$$M_{n\ell} = \frac{2(0.812)(33)(1{,}000)}{12} = 4{,}466 \text{ ft-lb}$$

$$\frac{M_{n\ell}}{\Omega_b} = \frac{4{,}466 \text{ ft-lb}}{1.67} = 2{,}674 \text{ ft-lb}$$

LRFD option:

$$M_{n\ell} = S_e F_n \qquad \text{AISI Eq. F3.1-1}$$

Since header is fully braced, $F_n = F_y$

$$M_{n\ell} = \frac{2(0.812)(33)(1{,}000)}{12} = 4{,}466 \text{ ft-lb}$$

$$\phi_b M_{n\ell} = 0.90(4{,}466) = 4{,}019 \text{ ft-lb}$$

THE CORRECT ANSWER IS: (B)

Solution 125, Vertical Forces, p. 84
The solution should read as follows:

$$d_e = d_s = 60 - 1.5 - \frac{1.128}{2} - 0.625 = 57.31 \text{ in.} \qquad \text{AASHTO 5.3}$$

$$a = \frac{A_s f_y}{\alpha_1 f'_c b} = \frac{10 \times 1.00 \times 60}{0.85 \times 4 \times 36} = 4.902 \text{ in.} \qquad \text{AASHTO 5.6.2.2}$$

$$\therefore d_v = d_e - \frac{a}{2} = 57.31 - \frac{4.902}{2} \qquad \text{AASHTO 5.3}$$

$$= 54.86 \text{ in.} \quad \text{Controls}$$

$$0.9\, d_e = 51.58 \text{ in.} < 54.86 \text{ in.} \qquad \text{AASHTO 5.7.2.8}$$

$$0.72\, h = 43.2 \text{ in.} < 54.86 \text{ in.}$$

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Question 124, Lateral Forces, p. 163
The solution should read as follows:

An office building is supported by special concentrically braced frames.

Design Codes:
IBC: *International Building Code*, 2018 edition.
AISC: *Seismic Design Manual,* 3rd edition.

Design Data:

Seismic Design Category	D
Hollow structural section tubes	A500 Grade C

Assumption:
Amplified seismic brace force = 175 kips.

The required tensile strength of the bracing connection (kips) is most nearly:

	ASD	LRFD
(A)	60	90
(B)	80	120
(C)	100	150
(D)	120	175

Question 125, Lateral Forces, p. 164
The options should read as follows:

(A) 71
(B) 142
(C) 155
(D) 219

Question 128, Lateral Forces, p. 167
The options should read as follows:

(A) 30
(B) 36
(C) 46
(D) 60

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Question 801, Lateral Forces—Buildings, p. 167
The following design data was added:

Design Data:

Wind	Basic wind speed = 142 mph
	Exposure C
	K_{zt} = 1.67
	K_e = 1.0

Solution 124, Lateral Forces, p. 217
R_y = 1.3 — AISC SDM Table A3.1
F_y = 50 ksi — AISC SDM Table 1-5b

ASD option: — AISC SDM Sec. F2.6c(1)

$$\frac{R_y F_y A_g}{1.5} = \frac{(1.3)(50\text{ ksi})(2.24\text{ in}^2)}{1.5} = 97.1\text{ kips}$$

LRFD option:

$$R_y F_y A_g = (1.3)(50\text{ ksi})(2.24\text{ in}^2) = 145.6\text{ kips}$$

Solution 125, Lateral Forces, p. 217
The solution should read as follows:

The required tensile strength is the lesser of:
a. The expected yield strength, in tension, of the brace: $R_y F_y A_g$ — AISC SDM Sec. F2.6c(1)
b. The maximum load effect that can be transferred to the system

R_y = 1.3 HSS ASTM A500 Grade C — AISC SDM Table A3.1
A_g = 3.37 in^2
$R_y F_y A_g$ = (1.3)(50 ksi)(3.37 in^2)
= 219 kips

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Solution 128, Lateral Forces, p. 220
The solution should read as follows:

$$l_d = l_{db} \times \left(\frac{\lambda_{rl}\lambda_{cf}\lambda_{rc}\lambda_{er}}{\lambda} \right)$$ AASHTO 5.10.8.2.1a-1

$$l_{db} = 2.4 d_b \frac{f_y}{\sqrt{f'_c}};\ d_b = 1.27",\ f_y = 60 \text{ ksi},\ f'_c = 4 \text{ ksi}$$ AASHTO 5.10.8.2.1a-2

$$= 2.4(1.27)\frac{60}{\sqrt{4}} = 91.4"$$

$$\lambda_{rl} = 1.0;\ \lambda_{cf} = 1.0(\text{not coated});\ \lambda = 1.0;\ \text{assume } \lambda_{er} = 1.0$$

$$\lambda_{rc} = \frac{d_b}{c_b + K_{tr}} = \frac{1.27"}{c_b + K_{tr}}$$

c_b = smaller of (center of bar to face of member or 1/2 c/c bar sp.)

$$c_b = \min\left[2" \text{ cover} + 0.5" \text{ spiral} + 1/2\ (1.27" \text{ bar}),\ 1/2(5" \text{ spacing})\right]$$

$$c_b = 2.5"$$

$$\therefore \lambda_{rc} = \frac{1.27"}{2.5 + 2} = 0.28 < 0.4; \text{ therefore, use } 0.4$$

$$K_{tr} = 40A_{tr}/(sn)$$

$$A_{tr} = 0.2 \text{ in}^2;\ s = 4";\ n = 1$$

$$K_{tr} = 40(0.2)/4(1) = 2$$

$$\therefore l_d = 91.4"\left(\frac{1.0 \times 1.0 \times 0.4 \times 1.0}{1.0}\right)$$

$$= 36.6" \times 1.25 = 45.8"$$ AASHTO 5.11.4.3

THE CORRECT ANSWER IS: (C)

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Solution 801, Lateral Forces—Buildings, p. 228
The following was added under the (b) in the solution:

(b) Design wind pressure on the parapet ASCE 7 Sec. 30.8

$p = q_p(GC_p - GC_{pi})$ (Components and cladding elements of parapets)

$q_p = 0.00256\,K_zK_{zt}K_dK_eV^2$ ASCE 7 Eq. 26.10-1

$K_z = 0.90$ Exposure C, $z = 20$ ft (top of parapet) ASCE 7 Table 26.10-1

$K_{zt} = 1.67$ (given)

$K_e = 1.0$ (given)

$K_d = 0.85$ ASCE 7 Table 26.6-1

$V = 142$ mph (given)

$q_p = 0.00256 \times 0.90 \times 1.67 \times 0.85 \times 142^2 = 65.9$ psf

$GC_{pi} = 0.00$ (solid parapet, open building condition) ASCE 7 Table 26.13-1

GC_p, $h \le 60$ ft ASCE 7 Fig. 30.3-1

Effective wind area ASCE 7 Sec. 26.2

= span length 4-ft height × span 4 ft/3 = 16/3 = 5.3 ft^2 < 10 ft^2 Use 10 ft^2

Zone 4 wall positive pressure $GC_p = +1.0 \times 0.90 = +0.90$ (Ref footnote 5 for reduction)

Zone 4 wall negative pressure $GC_p = -1.1 \times 0.90 = -0.99$ (Ref footnote 5 for reduction)

Zone 2 roof negative pressure $GC_p = -2.3$ ASCE 7 Fig. 30.3-2A

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Solution 903, Lateral Forces—Bridges, p. 260
The following was adjusted under the Procedure 2 in the solution:

According to Table 3.4.1-1 for Extreme Event I:

1.0DC + 1.0EQ

The above equation requires combining seismic Load Cases 1 and 2 with the dead load forces as given in the problem statement.

DC = dead load axial force in column of Pier 2 = 664 kips (given)

According to Table 3.10.7.1-1, Art. 3.10.7.1, the response modification factor R for multiple column bents with an importance category of "Other" is 5. The R factor for column shear force is 1 per Art.3.10.9.4.3d. The seismic moments should be divided by R and combined with the dead load moments. The following table shows the modified elastic design moments.

Determine the maximum forces:

Force	Load Case 1 **1.0(DC) ± 1.0(EQ)**			Load Case 2 **1.0(DC) ± 1.0(EQ)**		
V_L	1.0(0) + 1.0(810)	=	810 kips	1.0(0) +1.0(243)	=	243 kips
V_T	1.0(21) + 1.0(243)	=	264 kips	1.0(21) + 1.0(810)	=	831 kips
P_{max}	1.0(664) + 1.0(287)	=	951 kips	1.0(664) + 1.0(957)	=	1,621 kips
M_L	1.0(0) + 1.0(8,100/5)	=	1,620 ft-kips	1.0(0) + 1.0(2,430/5)	=	486 ft-kips
M_T	1.0(162) + 1.0(2,430/5)	=	648 ft-kips	1.0(162) + 1.0(8,100/5)	=	1,782 ft-kips

The above table indicates that the combination for Load Case 2 governs.

$$M_u = \text{factored moment} = (486^2 + 1,782^2)^{1/2} = 1,847 \text{ ft-kips}$$

$$V_u = \text{factored shear} = (243^2 + 831^2)^{1/2} = 866 \text{ kips}$$

The summary of maximum design forces at the base of each column of Pier 2 is as follows:

M_u = 1,847 ft-kips

V_u = 866 kips

P_u = 1,621 kips