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

![Diagram](image)

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

ASCE 7, Figure 7.6-1 Formula.
\[ \frac{h_d}{I_s} = \left( \frac{0.43}{l_u} \frac{\sqrt{p_g + 10}}{4} \right) - 1.5 \]
\[ p_g = 30, \quad l_u = 100 \text{ ft} \quad \text{given) \quad \text{Risk Category IV} \quad \therefore I_s = 1.20 \quad \text{ASCE 7 Table 1.5-2} \]
\[ h_d = \left[ \left( 0.43 \sqrt{100} \frac{\sqrt{30 + 10}}{4} \right) - 1.5 \right] \left( \sqrt{1.20} \right) = 3.86 \text{ ft} \]

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

Strength I, \( M_a = 1.25(25.8 \text{ ft-kips}) + 1.75(492.7 \text{ ft-kips}) \quad \text{AASHTO Table 3.4.1-1} \]
\[ = 894.5 \text{ ft-kips} \]

THE CORRECT ANSWER IS: (B)
Solution 124, Vertical Forces, p. 83
The solution should read as follows:

ASD option:
\[ M_{ne} = S_c F_n \]  
\[ \text{AISI Eq. F3.1-1} \]

Since header is fully braced, \( F_n = F_y \)
\[ M_{ne} = \frac{2(0.812)(33)(1,000)}{12} = 4,466 \text{ ft-lb} \]
\[ \frac{M_{ne}}{\Omega_b} = \frac{4,466 \text{ ft-lb}}{1.67} = 2,674 \text{ ft-lb} \]

LRFD option:
\[ M_{ne} = S_c F_n \]  
\[ \text{AISI Eq. F3.1-1} \]

Since header is fully braced, \( F_n = F_y \)
\[ M_{ne} = \frac{2(0.812)(33)(1,000)}{12} = 4,466 \text{ ft-lb} \]
\[ \phi_b M_{ne} = 0.90(4,466) = 4,019 \text{ ft-lb} \]

THE CORRECT ANSWER IS: (B)

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

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

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

\[ d_v = \frac{a}{2} = \frac{57.31 - 4.902}{2} \]
\[ = 54.86 \text{ in.} \]  
\[ \text{AASHTO 5.3} \]

\[ 0.9 d_e = 51.58 \text{ in.} < 54.86 \text{ in.} \]  
\[ 0.72 h = 43.2 \text{ in.} < 54.86 \text{ in.} \]  
\[ \text{AASHTO 5.7.2.8} \]
Question 124, Lateral Forces, p. 163
The solution should read as follows:

An office building is supported by special concentrically braced frames.

Design Codes:

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:

<table>
<thead>
<tr>
<th></th>
<th>ASD</th>
<th>LRFD</th>
</tr>
</thead>
<tbody>
<tr>
<td>(A)</td>
<td>60</td>
<td>90</td>
</tr>
<tr>
<td>(B)</td>
<td>80</td>
<td>120</td>
</tr>
<tr>
<td>(C)</td>
<td>100</td>
<td>150</td>
</tr>
<tr>
<td>(D)</td>
<td>120</td>
<td>175</td>
</tr>
</tbody>
</table>

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
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
F_y = 50 ksi

**ASD option:**
\[ \frac{R_y F_y A_g}{1.5} = \frac{(1.3)(50 \text{ ksi})(2.24 \text{ in}^2)}{1.5} = 97.1 \text{ kips} \]

**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: \( R_y F_y A_g \)

b. The maximum load effect that can be transferred to the system

\[ R_y = 1.3 \quad \text{HSS ASTM A500 Grade C} \]

\[ A_g = 3.37 \text{ in}^2 \]

\[ R_y F_y A_g = (1.3)(50 \text{ ksi})(3.37 \text{ in}^2) = 219 \text{ kips} \]
Solution 128, Lateral Forces, p. 220
The solution should read as follows:

\[ l_d = l_{db} \times \left( \frac{\lambda_r \lambda_{cf} \lambda_{rc} \lambda_{er}}{\lambda} \right) \]

\[ l_{db} = 2.4d_b \frac{f_y}{\sqrt{f_c'}} \quad ; \quad d_b = 1.27" \quad , \quad f_y = 60 \text{ ksi}, \quad f_c' = 4 \text{ ksi} \]

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

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

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

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

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

\[ c_b = 2.5" \]

\[ \therefore \lambda_{rc} = \frac{1.27"}{2.5 + 2} = 0.28 < 0.4; \quad \text{therefore, use } 0.4 \quad K_{tr} = 40A_{tr} / (sn) \]

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

\[ K_{tr} = 40(0.2) / 4(1) = 2 \]

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

\[ = 36.6" \times 1.25 = 45.8" \quad \text{AASHTO 5.11.4.3} \]

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

(b) Design wind pressure on the parapet

\[ p = q_p (G_{C_p} - G_{C_{pi}}) \]

Components and cladding elements of parapets

\[ q_p = 0.00256 K_z K_{zt} K_d K_e V^2 \]

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)

\[ q_p = 0.00256 \times 0.90 \times 1.67 \times 0.85 \times 142^2 = 65.9 \text{ psf} \]

ASCE 7 Table 26.13-1

GC\_pi = 0.00 (solid parapet, open building condition)

ASCE 7 Fig. 30.3-1

GC\_p, h \leq 60 ft

Effective wind area

\[ = \text{span length 4-ft height} \times \text{span 4 ft/3} = 16/3 = 5.3 \text{ ft}^2 < 10 \text{ ft}^2 \text{ Use 10 ft}^2 \]

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
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:

<table>
<thead>
<tr>
<th>Force</th>
<th>Load Case 1 1.0(DC) ± 1.0(EQ)</th>
<th>Load Case 2 1.0(DC) ± 1.0(EQ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_L )</td>
<td>( 1.0(0) + 1.0(810) )</td>
<td>( 1.0(0) + 1.0(243) )</td>
</tr>
<tr>
<td>( V_T )</td>
<td>( 1.0(21) + 1.0(243) )</td>
<td>( 1.0(21) + 1.0(810) )</td>
</tr>
<tr>
<td>( P_{max} )</td>
<td>( 1.0(664) + 1.0(287) )</td>
<td>( 1.0(664) + 1.0(957) )</td>
</tr>
<tr>
<td>( M_L )</td>
<td>( 1.0(0) + 1.0(8,100/5) )</td>
<td>( 1.0(0) + 1.0(2,430/5) )</td>
</tr>
<tr>
<td>( M_T )</td>
<td>( 1.0(162) + 1.0(2,430/5) )</td>
<td>( 1.0(162) + 1.0(8,100/5) )</td>
</tr>
</tbody>
</table>

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:

\[ M_u = 1,847 \text{ ft-kips} \]

\[ V_u = 866 \text{ kips} \]

\[ P_u = 1,621 \text{ kips} \]