ERRATA for
Structural Engineering Practice Exam
ISBN 978-1-932613-69-8

Errata posted 11-7-2016

Revisions are shown in red type.

Vertical Forces PM Bridges

Solution 703, pp. 119–124

Solution 703 should read as follows on page 119:

703. **(a) Verify the plate size for the flange splice.**

Per AASHTO Section 6.13.2.1, the connection must be checked for slip resistance and the shear and bearing resistance checked separately.

Calculate effective flange area—tension flange:

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \qquad \text{Eq. 6.13.6.1.4c-2}$$

Deducted flange width for bolt hole — Art. 6.8.3

Hole diameter = 15/16 in. — Table 6.13.2.4.2-1

$$\begin{aligned} W_n &= 16.5 - (4)(5/16) + (2)(3.5)^2/[(4)(3.5)] \\ &= 14.5 \text{ in.} \\ A_n &= (14.5)(1.26) \\ &= 18.27 \text{ in}^2 \\ A_g &= b_f \times t_f \\ &= (16.5)(1.26) \\ &= 20.79 \text{ in}^2 \end{aligned}$$

$\phi_u = 0.80$ — Art. 6.5.4.2

$\phi_y = 0.95$

$F_u = 58$ — Table 6.4.1-1

$F_{yt} = 36$

$$A_e = \frac{(0.8)(58)}{(0.95)(36)}\left(18.27 \text{ in}^2\right)$$

$$= (1.357)\left(18.27 \text{ in}^2\right) = 24.79 \text{ in}^2 > A_g = 20.79 \text{ in}^2 \text{ Governs.}$$

The gross section properties can be used.

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Determine the controlling flange:

Check each flange for stress at the midpoint of the flange due to tension forces from applied loads.

At the bottom flange (Dead Load + Maximum Positive Live Load):

Tables 3.4.1-1 & 2

$$\begin{aligned} M &= 0.9\,DC + 1.5\,DW + 1.75\,(LL + I) \\ &= 0.9(-69) + 1.5(26) + 1.75(730) \\ &= 1{,}254.4 \text{ ft-kips} \end{aligned}$$

$S_X = I_X/C$ At **center** of beam flange

$$\begin{aligned} &= 15{,}600/[(36.5 - 1.26)/2] \\ &= 885 \text{ in}^3 \end{aligned}$$

Solution 703 should read as follows on page 122:

Check splice plate size:

Outside plate:

$$\begin{aligned} W_n &= 16.0 - (4)(15/16) + (2)(3.5)^2/[(4)(3.5)] \\ &= 14.0 \text{ in.} \end{aligned}$$

$A_n = 14.0 \times 5/8" = 8.75 \text{ in}^2 > 0.85\,A_g$ Governs.

$0.85\,A_g = 0.85(10) = 8.5 \text{ in}^2$ Art. 6.13.5.2

$\therefore A_n = 8.5 \text{ in}^2$

Inside plate:

$$\begin{aligned} W_n &= 2\left\{6.5 - (2)(15/16) + (3.5)^2/[(4)(3.5)]\right\} \\ &= 11.0 \text{ in.} \end{aligned}$$

$A_n = 11.0 \times 5/8" = 6.88 \text{ in}^2 < 0.85\,A_g$

$0.85\,A_g = 0.85(8.13) = 6.91$ OK

$\therefore A_n = 6.88 \text{ in}^2$

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For tension:
For yielding: Eq. 6.8.2.1-1

Outside $P_r = \phi_y F_y A_g = (0.95)(36\text{ ksi})(10\text{ in}^2) = 342\text{ kips} > 309.6\text{ kips}$ OK

Inside $P_r = \phi_y F_y A_g = (0.95)(36\text{ ksi})(8.13\text{ in}^2) = 278.0\text{ kips} > 251.7\text{ kips}$ OK

For fracture: Eq. 6.8.2.1-2

Outside $P_r = \phi_u F_u A_n U = (0.8)(58\text{ ksi})(8.5\text{ in}^2)(1.0) = 394.4\text{ kips} > 309.6\text{ kips}$ OK

Inside $P_r = \phi_u F_u A_n U = (0.8)(58\text{ ksi})(6.88\text{ in}^2)(1.0) = 319.2\text{ kips} > 251.7\text{ kips}$ OK

For compression:
$R_r = \phi_c F_y A_s$ where $A_s = A_g$ Eq. 6.13.6.1.4c-4

Outside $R_r = (0.9)(36\text{ ksi})(10) = 324\text{ kips} > 309.6\text{ kips}$ OK

Inside $R_r = (0.9)(36\text{ ksi})(8.13) = 263.4\text{ kips} > 251.7\text{ kips}$ OK

Solution 703 should read as follows on p. 123.

(b) Verify the number of bolts in the flange splice, and revise the number if required.

Check flange bolts:

Maximum distance between end fasteners = 3.5 in. × 7 spaces = 24.5 in. < 50 in. so 20% decrease in bolt strength per Art. 6.13.2.7 is not required.

By specification, the bolt threads are included in the shear plane; hence the bolt strength from Eq. 6.13.2.7-2 will be used directly.

Bolt shear strength:

$R_n = 0.38\, A_b F_{ub}\, N_s$ Eq. 6.13.2.7-2

$F_{ub} = 120\text{ ksi}$ Art. 6.4.3.1

$R_n = 0.38(0.6)(120)(1) = 27.4\text{ kips}$

$\phi R_n = 0.8(27.4)$ where $\phi_s = 0.80$

$= 21.9$ kips per bolt Bolt strength controls over bearing strength.

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Bolt bearing strength:

F_u = 58 ksi M-270, Gr 36 steel
L_c = 2 – (1-in.-dia./2) Clear distance from edge of hole to edge of connected plate
= 1.5 in. < 2d = 1.75 in.
d = 7/8 in. Nominal bolt diameter
t = 5/8 in. or 1.26 in. Thickness of connected material
$R_n = 1.2L_ctF_u$ Eq. 6.13.2.9-2

Bearing on splice plates:

ϕR_n = (0.8)(1.2)(1.5)(5/8)(58) where ϕ_{bb} = 0.8
= 52.2 kips per bolt Does not control bolt strength

Bearing on beam flange:

ϕR_n = (0.8)(1.2)(1.5)(1.26)(58)
= 105.2 kips per bolt Does not control bolt strength

Required number of bolts:

At outside plate = (21.9 kips per bolt)(16 bolts) Single shear per plate
= 350.4 kips > 309.6 kips OK

At inside plates = (21.9 kips per bolt)(16 bolts) Single shear per plate
= 350.4 kips > 251.7 kips OK

Previously posted errata continued on next page.

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Vertical Forces AM

Question 123, p. 31

The design data should read as follows:

123. Design Data:

Weight of girder	822 plf
Prestressing force at release	650 kips
Area of girder	789 in^2
Section moduli for the girder:	
Top fiber	8,089 in^3
Bottom fiber	10,543 in^3

Previously posted errata continued on next page.

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Errata posted 2-5-2016

Lateral Forces AM
Question 121, p. 151

The options and the drawing shown as follows:

121. The figure shows a braced frame connection at the beam/brace location.

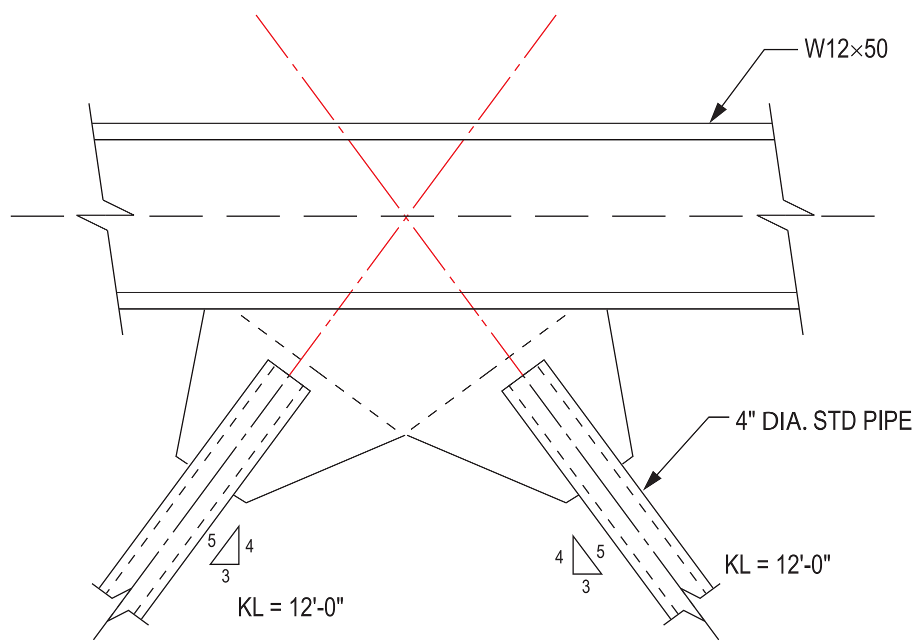
Design Code:
AISC: *Seismic Design Manual*, 2nd edition.

Design Data:
ASTM A53 pipe steel braces.

Assumption:
Special concentrically braced frame designed per AISC *Seismic Design Manual*.

The vertical portion of the earthquake effect E (kips) in the beam at the point of the connection is most nearly:

(A) 111
(B) 155
(C) 166
(D) 255



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Errata posted 2-5-2016

Solution 121, p. 197
The solution should read as follows:

121. The beam is required to resist the force from one brace in tension and 30% of one brace in compression. SDM 2nd edition, AISC 341, Sec. F2.3

Expected brace strength in tension:

$R_yF_yA_g$

$R_y = 1.6$ SDM 2nd edition, AISC 341, Table A3.1

$F_y = 35\ \text{ksi}$ SDM 2nd edition, AISC 341, Table 1-7

A_g of 4-in.-dia. STD pipe $= 2.96\ \text{in}^2$

$R_yF_yA_g = (1.6)(35)(2.96) = 166\ \text{kips}$

Vertical component $= \frac{4}{5}(166\ \text{kips}) = 133\ \text{kips}$

Expected braced strength in compression:

Lesser of $R_yF_yA_g$ (166 kips) and 1.14 F_{cre} A_g

$F_{cre} = F_{cr}$ using R_yF_y for F_y

r of 4 in.-dia. STD pipe = 1.51 in.

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121. (Continued)

$$\frac{kL}{r} = \frac{(12\,\text{ft})(12)}{1.51\text{ in.}} = 95.4$$

$$4.71\sqrt{\frac{E}{R_yF_y}} = 4.71\sqrt{\frac{29{,}000\text{ ksi}}{1.6(35\text{ ksi})}} = 107.2$$

$$\therefore F_{cre} = \left[0.658^{\frac{R_yF_y}{F_e}}\right]R_yF_y$$ AISC, 14th edition, Eq. E3-2

$$F_e = \frac{\pi^2 E}{\left(\frac{kL}{r}\right)^2}$$ AISC, 14th edition, Eq. E3-4

$$F_e = \frac{\pi^2(29{,}000\text{ ksi})}{(95.4)^2} = 31.4\text{ ksi}$$

$$F_{cre} = \left[0.658^{\frac{(1.6)(35\text{ ksi})}{31.4\text{ ksi}}}\right](1.6)(35\text{ ksi}) = 26.5\text{ ksi}$$

$$1.14\,F_{cre}A_g = 1.14(26.5\text{ ksi})(2.96\text{ in}^2) = 89\text{ kips}$$ Controls

$$0.3(1.14\,F_{cre}A_g) = 0.3(89\text{ kips}) = 27\text{ kips}$$

$$\text{Vertical component} = \frac{4}{5}(27\text{ kips}) = 22\text{ kips}$$

$$\therefore \text{The vertical portion of the earthquake effect, } E = 133 - 22 = 111\text{ kips}$$

THE CORRECT ANSWER IS: (A)

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Vertical Forces PM Buildings
Solution 602, pp. 100-102

The solution should read as follows:

602. (Continued)

Check weld required for flange PL to column:

Fillet weld size required for E70XX

$\phi R_n = \phi F_{nw} A_{we}$ AISC Eq. J2.4

$\phi = 0.75$

$F_{nw} = 0.60\, F_{EXX}\left(1.0 + 0.50 \sin^{1.5}\theta\right)$

$= 0.60\,(70)\left(1.0 + 0.50 \sin^{1.5} 90\right) = 63 \text{ ksi}$

$A_{we} = t_e \ell$

$= 0.707\,(D/16)(6\,3/4")(2)$

$= 0.597\,D$

$(0.75)(63)(0.597\,D) = 80 \text{ kips}$

$\therefore D = 2.8 \Rightarrow 3/16"$ fillet weld

AISC Table J2.4 requires min. 3/16" fillet weld for 3/8" thick material (thinner part joined).

$\therefore$ Use 3/16" fillet weld top and bottom of flange PL

For single shear PL web connection to column:

Try a PL $3/8" \times 4" \times 9"$ with (3) 3/4" dia. A325N bolts (Group A bolts)

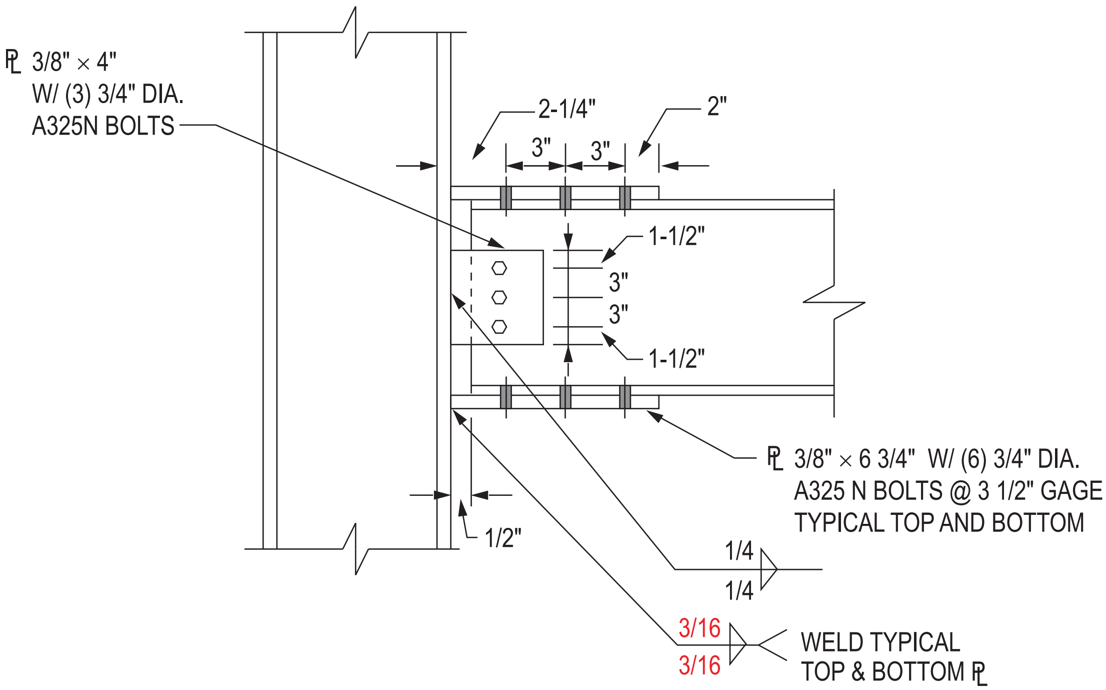
Per AISC Table 10-10a, LRFD available strengths = 43.4 kips > 37.8 kips OK

Per notes on p. 10-107, weld size = (5/8) t_p

$\therefore$ Weld size = $(5/8)(3/8) = 15/64 \Rightarrow$ use 1/4" fillet weld

602. **(Continued)**

(c) Sketch of connection detail



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Lateral Forces PM Buildings
Solution 803, pp. 215–217

The solution should read as follows:

803. **(Continued)**
Hooked dowel embedment:

$$\ell_{dh} = \frac{0.02\,\psi_e f_y}{\lambda\sqrt{f'_c}} d_b \qquad \text{ACI Sec. 12.5.2}$$

$$= \frac{0.02(1.0)(60,000)}{1.0\sqrt{3,000}}(5/8) = 13.7"$$

$$\ell_{dh} = 13.7" < 33" \text{ provided} \qquad \text{OK}$$

Previously posted errata continued on next page.

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Errata posted 8-15-2015

Lateral Forces PM Buildings

Question 801, p. 170

Requirement (c) should read as follows:

REQUIREMENTS:

(c) For a horizontal service level wind pressure of 100 psf, check whether #5 at 48" o.c. vertical reinforcement at the centerline of the wall is adequate for the parapet.

Solution 801, pp. 209–210

Requirement (c) should read as follows:

(c) Check if the existing #5 @ 48" o.c. vertical at centerline of wall is adequate for the parapet:

$$M_{max} = \frac{100\text{ psf }(1')(4')^2}{2} = 800\text{ ft-lb / ft wall}$$

Check flexural capacity:

$$A_s = 0.31\text{ in}^2\ (12/48) = 0.0775\text{ in}^2\text{ / ft}$$

$$\rho = \frac{0.0775}{(12)(7.625/2)} = 0.0017$$

$$\eta = \frac{E_s}{E_m} = \frac{29\times10^6}{900\ (1{,}500)} = 21.48$$

$$\eta\rho = (21.48)(0.0017) = 0.037$$

$$k = \sqrt{\eta\rho^2 + 2\eta\rho} - \eta\rho = 0.238$$

$$j = 1 - k/3 = 0.921$$

$$f_b = \frac{2\text{ m}}{kjbd^2} = \frac{(2)(800)(12)}{(0.238)(0.921)(12)(7.625/2)^2} = 502\text{ psi}$$

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801. (Continued)

$F_b = 0.45 f'_m = 0.45(1{,}500) = 675$ psi TMS 402 Sec 2.3.4.2.2

$F_b > f_b$ OK

$$f_s = \frac{m}{A_s jd} = \frac{(800)(12)}{(0.0775)(0.921)(7.625/2)} = 35{,}278 \text{ psi}$$

$F_s = 32{,}000$ psi TMS 402 Sec 2.3.3.1(b)

$F_s < f_s$ No good

$\therefore$ #5 @ 48" o.c. in 8" CMU parapet wall (partially grouted) is inadequate.

Previously posted errata continued on next page.

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Lateral Forces PM Buildings
Question 801, p. 171

Question 801 figure should read as follows:

801. **(Continued)**

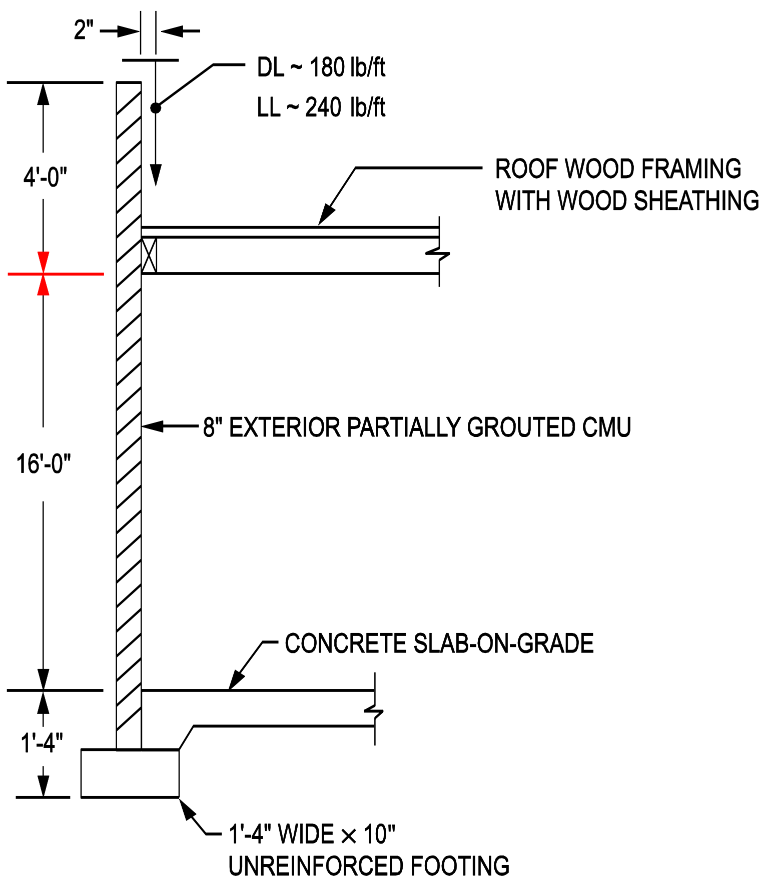

FIGURE 801

Previously posted errata continued on next page.

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Lateral Forces AM
Question 138, p. 167

Question 138 should read as follows:

138. The figure shows column pile caps interconnected by a grade beam that acts as a seismic tie.

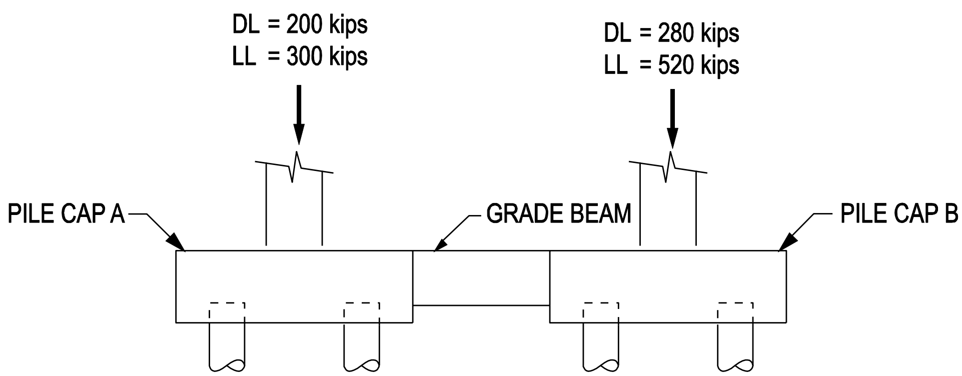
Design Code:
IBC: *International Building Code,* 2012 edition (without supplements).

Design Data:
Seismic Design Category D
S_{DS} = 0.75 g

Assumption:
Ignore weight of pile cap

The design strength force P (kips) to be resisted by the grade beam in tension or compression is most nearly:

(A) 25
(B) 60
(C) 88
(D) 180



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Errata posted 4-10-2015

Lateral Forces AM
Solution 138, p. 204

Solution 138 should read as follows:

Pile Cap A
$1.2\,D + 1.6\,L = 1.2(200) + 1.6(300) = 720$ kips
Pile Cap B
$1.2\,D + 1.6\,L = 1.2(280) + 1.6(520) = 1{,}168$ kips
Seismic tie tension or compression IBC 2012, Sec. 1810.3.13

$$T = C = \frac{1{,}168 \times S_{DS}}{10} = \frac{1{,}168(0.75)}{10} = 87.6 \text{ kips}$$

$T = C = 0.25 \times 720 = 180$ kips
$\therefore\ T = C = 88$ kips

THE CORRECT ANSWER IS: (C)

Previously posted errata continued on next page.

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Lateral Forces AM Solutions
Solution 115, pp. 194–195

Solution 115 should read as follows:

115. ASD solution:

ASD load combinations per ASCE 7-10, Section 2.4.1 involving D, L + W:

Taking counterclockwise moment as positive at Node 2

#7: $0.6\,D + 0.6\,W$ (uplift condition of $R_1 = -6.3$, assuming F_W acts to right)

$$\Sigma M_2 = 0.6(D)(20\text{ ft})(10\text{ ft}) - 0.6F_W(15\text{ ft}) + R_1(20\text{ ft}) = 0$$

$$0 = 0.6(200\text{ plf})(20\text{ ft})(10\text{ ft}) - 0.6F_W(15\text{ ft}) + (6{,}300\text{ lb})(20\text{ ft})$$

$$0.6\,F_W = 10.0\text{ kips (acting to right)}$$

#5: $D + 0.6\,W$ (bearing condition of $R_1 = +9.5$, assuming F_W acts to left)

$$\Sigma M_2 = (D)(20\text{ ft})(10\text{ ft}) + 0.6\,F_W(15\text{ ft}) - R_1(20\text{ ft}) = 0$$

$$0 = (200\text{ plf})(20\text{ ft})(10\text{ ft}) + (0.6\,F_W)(15\text{ ft}) - (9{,}500\text{ lb})(20\text{ ft})$$

$$0.6\,F_W = 10\text{ kips (acting to left)}$$

#6a: D + 0.75 L + 0.75 (0.6 W) (bearing condition of $R_1 = +9.5$, assuming F_W acts to left)

$$\Sigma M_2 = (D + 0.75\,L)(20\text{ ft})(10\text{ ft}) + 0.75(0.6\,F_W)(15\text{ ft}) - R_1(20\text{ ft}) = 0$$

$$0 = \left[200\text{ plf} + 0.75(200\text{ plf})\right](20\text{ ft})(10\text{ ft}) + 0.75(0.6\,F_W)(15\text{ ft}) - (9{,}500\text{ lb})(20\text{ ft})$$

$$0.6\,F_W = 10.7\text{ kips (acting to left)}$$

$\therefore$ since $0.6\,F_W$ acts in both directions and must be identical values, $0.6\,F_W$ = 10 kips to generate maximum and minimum R_1 values given.

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115. (Continued)

LRFD solution:

LRFD load combinations per ASCE 7-10, Section 2.3.2 involving D, L + W:

Taking counterclockwise moment as positive at Node 2:

#6: $0.9\,D + 1.0W$ (uplift condition of R_1 = -10.7, assuming F_W acts to right)

$$\Sigma M_2 = 0.9(D)(20\text{ ft})(10\text{ ft}) - (F_W)(15\text{ ft}) + R_1\,(20\text{ ft}) = 0$$

$$0 \quad = 0.9(200\text{ plf})(20\text{ ft})(10\text{ ft}) - (F_W)(15\text{ ft}) + (10{,}700\text{ lb})\,(20\text{ ft})$$

$$F_W = 16.7\text{ kips} \therefore 0.6 F_W = 10.0\text{ kips (acting to right)}$$

#3: $1.2\,D + 0.5\,W$ (bearing condition of R_1 = $+16.9$, assuming F_W acts to left)

$$\Sigma M_2 = 1.2(D)(20\text{ ft})(10\text{ ft}) + 0.5 F_W (15\text{ ft}) - R_1 (20\text{ ft}) = 0$$

$$0 \quad = (1.2)(200)(20\text{ ft})(10\text{ ft}) + 0.5 F_W (15\text{ ft}) - (16{,}900)(20\text{ ft}) = 0$$

$$F_W = 38.7 \quad \therefore 0.6\; F_W = 23.2\text{ kips (acting to left)}$$

#4: $1.2\,D + 1.0\,L + 1.0\,W$ (bearing condition of R_1 = $+16.9$, assuming F_W acts to left)

$$\Sigma M_2 = \left[1.2(D) + L)\right](20\text{ ft})(10\text{ ft}) + F_W (15\text{ ft}) - R_1\,(20\text{ ft}) = 0$$

$$0 \quad = \left[(1.2)(200) + (200)\right](20\text{ ft})(10\text{ ft}) + F_W (15\text{ ft}) - (16{,}900)(20\text{ ft}) = 0$$

$$F_W = 16.7\text{ kips} \therefore 0.6\; F_W = 10.0\text{ kips (acting to left)}$$

$\therefore$ Since 0.6 F_W acts in both directions and must be identical values, 0.6 F_W = 10 kips to generate maximum and minimum R_1 values given.

THE CORRECT ANSWER IS: (B)

Previously posted errata continued on next page.

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Vertical Forces AM Solutions
Solution 121, pp. 78–79

Solution 121 should read as follows:

121. For a W14 × 53 column, d = 13.9 in. and b_f = 8.06 in. p. 1-24

From p. 14–5:

$$m = \frac{16 - 0.95\,(13.9)}{2} = 1.40 \qquad \text{Eq. 14-2}$$

$$n = \frac{10 - 0.8\,(8.06)}{2} = 1.78 \qquad \text{Eq. 14-3}$$

$$n' = \frac{\sqrt{13.9 \times 8.06}}{4} = 2.65 \qquad \text{Eq. 14-4}$$

ASD option:

$$\frac{P_n}{\Omega_c} @ 12\text{ ft} = 310\text{ kips} \qquad \text{p. 4-16}$$

$$\frac{4\,db_f}{(d+b_f)^2} = 0.929 \qquad \text{Eq. 14-6b}$$

$$X = 0.929 \times \frac{150}{310} = 0.450$$

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} = 0.770 \qquad \text{Eq. 14-5}$$

$$\lambda n' = 0.770 \times 2.65 = 2.04\text{ in.}$$

$$\ell = \text{larger }(1.40,\ 1.78,\ 2.04) = 2.04\text{ in.}$$

$$t_{\min} = \ell\sqrt{\frac{3.33P_a}{F_y BN}} = 2.04\sqrt{\frac{3.33(150)}{36(10)(16)}} \qquad \text{Eq. 14-7b}$$

$$= 0.60\text{ in.} \rightarrow \text{use } 5/8\text{ in.}$$

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Errata posted 12-19-14

121. (Continued)

LRFD option:

$\phi_c P_n$ @ 12 ft $= 465$ kips — p. 4-16

$$\frac{4db_f}{(d+b_f)^2} = 0.929 \qquad \text{Eq. 14-6a}$$

$$X = 0.929 \times \frac{190 \text{ kips}}{465 \text{ kips}} = 0.380$$

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} = 0.690$$

$$\lambda n' = 0.690 \times 2.65 = 1.83$$

$$\ell = \text{larger } (1.40, 1.78, 1.83) = 1.83 \text{ in.}$$

$$t_{\min} = \ell\sqrt{\frac{2\,P_u}{0.9\,F_y BN}} = 1.83\sqrt{\frac{2\,(190 \text{ kips})}{(0.9)(36 \text{ ksi})(10)(16)}} \qquad \text{Eq. 14-7a}$$

$$= 0.495 \rightarrow \text{Use } 1/2 \text{ in.}$$

THE CORRECT ANSWER IS: (C)