ERRATA for
Structural Engineering Sample Questions and Solutions
ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Revisions are shown in red.

Question 120, p. 28
The ASD option in line 10 should read as follows:

M_{nx}/Ω_b = 173 ft-kips

Question 121, p. 29
The column should be specified as follows:

The figure shows a W14×53 column and base plate.

Design Code:
AISC: *Steel Construction Manual,* 14th edition.

Design Data:

Base plate	F_y = 36 ksi
W14×53 column	F_y = 50 ksi
Compressive strength of concrete	f'_c = 3 ksi
Column axial load	150 kips (ASD) or 190 kips (LRFD)

Question 124, p. 32
The answer options should read as follows:

	ASD	LRFD
(A)	1,377	2,233
(B)	2,674	4,019
(C)	3,372	5,632
(D)	4,052	6,767

Question 134, p. 42
The answer options should read as follows:

	ASD	LRFD
(A)	665	705
(B)	1,250	1,300
(C)	1,465	1,550
(D)	2,335	2,475

ERRATA for
Structural Engineering Sample Questions and Solutions
ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Question 136, p. 44
The answer options should read as follows:

	ASD	LRFD
(A)	14,200	32,400
(B)	17,600	27,100
(C)	25,500	59,400
(D)	39,300	91,500

Question 604, p. 56
The design data should read as follows:

Design Data:

Existing glulam girder	8 1/2" × 26 1/8" Southern Pine stress class 24F-1.7E
New center post	8 × 8 Southern Pine No. 2
New steel plate	ASTM A36
New bolts	ASTM A307

New equipment load per **Figure 604C** (with new tension rods in place).
Properties of existing glulam girder are given in **Table 604.**

Question 702, p. 64
The assumptions should read as follows:
Bridge barrier and wearing surface are applied evenly to all girders.
Ignore design tandem loading.
Superstructure is a conventionally redundant system.
Bridge is considered operationally important.
Bridge is conventional design.

ERRATA for
Structural Engineering Sample Questions and Solutions
ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Solution 123, p. 82
The solution should read as follows:

Strength I, $M_u = 1.25(25.8 \text{ ft-kips}) + 1.75\,(492.7 \text{ ft-kips}) = 894.5 \text{ ft-kips}$ AASHTO 6.13

$F_{nc} = F_{nt} = R_h F_{yf}$ of smaller of the two sections ($W36\times135$, $S_x = 439 \text{ in}^3$)

$R_h = 1.0$ since all steel is 50 ksi; $F_n = 1.0(50 \text{ ksi}) = 50 \text{ ksi}$

$\phi_f = 1.00$ AASHTO 6.5.4.2

$f_{bu} \le \phi_f \cancel{R}_h^{0} F_{yf}$

$$f_{bu} = \frac{M_{rx}}{S_{xc}}$$

$$\frac{M_{rx}}{S_{xc}} \le \phi_f F_{yf}$$

$M_{rx} = \phi_f\ F_{yf} S_{xc} = 1.00\,(50 \text{ ksi})(439 \text{ in}^3)\,\frac{1}{12} = 1{,}829 \text{ ft-kips}$ AASHTO 6.10.8.1.3-1

Design connection for average $= \dfrac{894.5 \text{ ft-kips} + 1{,}829 \text{ ft-kips}}{2} = 1{,}362 \text{ ft-kips}$ AASHTO 6.13.1

or $= 0.75\,(1{,}829 \text{ ft-kips}) = 1{,}372 \text{ ft-kips}$ Controls

THE CORRECT ANSWER IS: (B)

Solution 124, p. 83

LRFD option:

$M_n = S_e F_y$ Eq. C3.1.1-1

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

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

THE CORRECT ANSWER IS: (B)

ERRATA for
Structural Engineering Sample Questions and Solutions
ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Solution 134, p. 89
Solution 134 should read as follows:

Working Stress Design

$f'_m = 2{,}000$ psi

$E_s = 29 \times 10^6$ Sec. 4.2.2.1

$E_m = 900\ f'_m$ Sec. 4.2.2.2.1

$E_m = 900(2{,}000) = 1.80 \times 10^6$ psi

$n = E_s/E_m = 29/1.80 = 16$

$$\rho = \frac{A_s}{bd} = \frac{0.31}{(48)\left(\frac{7.625}{2}\right)} = 0.0017$$

$n\rho = 0.0271$

$k = \sqrt{n\rho^2 + 2n\rho} - n\rho = 0.207$

$j = 1 - k/3 = 0.931$

$F_b = 0.45\ f'_m = 900$ psi Sec. 2.3.4.2.2

$$M_{max} = F_b bkjd^2/[2(12)] = 900(12)(0.207)(0.931)(7.625/2)^2/[2(12)]$$
$$= 1{,}260 \text{ ft-lb/ft}$$

ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Solution 134, p. 89 (Continued)

Strength design:

$\phi = 0.9$ 9.1.4.4

$$M_n = A_s f_y (d - a/2)$$

$$a = \frac{A_s f_y}{0.80 f'_m b}$$

$$M_n = \rho f_y b d^2 \left(1 - \frac{0.625\, \rho f_y}{f'_m}\right)$$

$$\rho = \frac{A_s}{bd}$$

$$\rho = 0.0017$$

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

$$= 0.08\ \text{in}^2/\text{ft}$$

$$M_n = (0.0017)(60)(12)(3.8125)^2 \left[1 - 0.625(0.0017)(60)/2\right]$$

$$= 17.2\ \text{in.-kips/ft}$$

$$= 1{,}433\ \text{ft-lb/ft}$$

$$\phi M_n = 1{,}289.7\ \text{ft-lb/ft}$$

THE CORRECT ANSWER IS: (B)

ERRATA for

Structural Engineering Sample Questions and Solutions

ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Solution 136, p. 91

The solution should read as follows:

Per Section 8.3.3.3, vertical bars must be laterally restrained. The #5 vertical reinforcing is not laterally restrained per Section 5.3.1.4; therefore analyze as an unreinforced masonry member.

ASD option:

$h = 144$ in. $r = 2.86$ in. $h/r = 50.3 < 99$

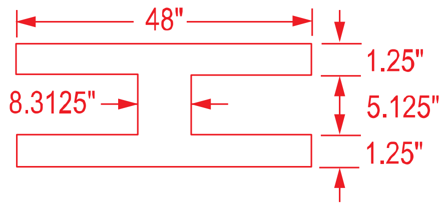
$$F_a = (1/4)\,f'_m\left[1-\left(\frac{h}{140\,r}\right)^2\right] \qquad \text{Eq. 8-21}$$

$$F_a = (1/4)(2{,}000)\left[1-\left(\frac{50.3}{140}\right)^2\right]$$

$F_a = 435$ psi

Net cross-sectional area of wall, A_n:

$A_n = (48 \times 1.25 \times 2) + (5.125 \times 8.3125) = 162.6\ \text{in}^2/4$ ft length of wall



$\therefore\ A_n = 40.65\ \text{in}^2/\text{ft}$

Design axial load $= F_aA_n = (435\ \text{psi})(40.65\ \text{in}^2/\text{ft})$

$= 17{,}683$ plf

LRFD option:

$h/r = 50.3$ (See ASD option) $\therefore\ h/r = < 99$

$$P_n = 0.80\ (0.80\ A_n f'_m\left[1-\left(\frac{h}{140\,r}\right)^2\right] \qquad \text{Eq. 9-11}$$

$$\phi = 0.60 \qquad \text{Sec. 9.1.4}$$

$A_n = 40.65\ \text{in}^2/\text{ft}$ (See ASD option)

$$\phi P_n = 0.60\left[0.80\left(0.80(40.65)(2{,}000)\left[1-\left(\frac{50.3}{140}\right)^2\right]\right)\right]$$

$\phi P_n = 27{,}189$ plf

THE CORRECT ANSWER IS: (B)

ERRATA for
Structural Engineering Sample Questions and Solutions
ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Solution 601, p. 101

(c) Wall-to-footing connection

Line 15 should read as follows:

However ℓ_{dh} not less than 8 db or 6 in. $\therefore$ Use $\ell_{dh} = 6$ in. ACI 318, Sec. 25.4.3.1

Solution 603, p. 108–109

(c) Design the beam for maximum positive moment in the span and negative moment and shear at column grid.

Positive moment (design as T-beam):

$M_u = 349$ ft-kips

Effective flange width: ACI 314 Sec. 6.3.2.1

$18 \text{ in.} + (2)(8)(9 \text{ in.}) = 162 \text{ in.}$

$\text{or } 18 \text{ in.} + \dfrac{342 \text{ in.}}{2} + \dfrac{333 \text{ in.}}{2} = 355.5 \text{ in.}$

$\text{or } 18 \text{ in.} + (2)\dfrac{324 \text{ in.}}{8} = 99 \text{ in.} (\text{controls})$

$d = 24 - (1.5 + 0.5 + 0.5) = 21.5 \text{ in.}$

From design aids:

$$M_u / \phi bd^2 = \frac{349 \times 12{,}000}{0.9 \times 99 \times 21.5^2} = 101.7$$

(Note: since the thickness of the slab flanges > a, design as rectangular section with tension reinforcing only)

From design aids table, $\rho = 0.0018$

$A_s = \rho bd = 0.0018 \times 99 \times 21.5 = 3.83 \text{ in}^2$

Min steel $A_{s\,min} = 3\sqrt{f'_c}\, b_w d/f_y = \dfrac{3\sqrt{4{,}000} \times 18 \times 21.5}{60{,}000}$ ACI 318 Sec. 9.6.1.2

$$= 1.22 \text{ in}^2 < 200\, b_w d/f_y = \frac{200 \times 18 \times 21.5}{60{,}000} = 1.29 \text{ in}^2$$

$\therefore A_{s\,required} = 3.83 \text{ in}^2$ Use (5) #8 bars $A_{s\,provided} = 3.95 \text{ in}^2$

ISBN 978-1-932613-89-6

Errata posted 4/29/2019

603. (Continued)

Negative moment at exterior face of first interior column:

$M_u = 507$ ft-kips b = 18 in. d = 21.5 in.

$$\frac{M_u}{\phi b d^2} = \frac{507 \times 12{,}000}{0.9 \times 18 \times 21.5^2} = 812.5$$

From design aids table, $\rho = 0.0158$

$$A_s = \rho b d = 0.0158 \times 18 \times 21.5 = 6.12 \text{ in}^2$$

$$\text{Min steel } A_{s\,min} = 3\sqrt{f'_c}\,(2b_w)\,d/f_y = \frac{3\sqrt{4{,}000} \times (2 \times 18) \times 21.5}{60{,}000}$$

$$= 2.45 \text{ in}^2 < 200\,(2b_w)\,d/f_y = \frac{200 \times (2 \times 18) \times 21.5}{60{,}000} = 2.58 \text{ in}^2$$

$\therefore A_{s\,required} = 6.12 \text{ in}^2$ Use (5) #10 bars $A_{s\,provided} = 6.35 \text{ in}^2$

Shear:

$V_u = 104.0$ kips

$$1/2\ \phi V_c = \frac{1}{2}\left(\phi 2 b d \sqrt{f'_c}\right) = \frac{0.75 \times 2 \times 18 \times 21.5 \times \sqrt{4{,}000}}{(2)(1{,}000)} = 18.4 \text{ kips} < 104.0 \text{ kips; Shear steel req.}$$

Use #4; $A_v = 2 \times 0.2 = 0.4 \text{ in}^2$

$$\text{Spacing } s = \frac{A_v f_{yt} d}{V_s/\phi} = \frac{0.4 \times 60 \times 21.5}{\left(\dfrac{104.0 - 18.4(2)}{0.75}\right)} = 5.76 \text{ in. Use 5 in.}$$

Maximum spacing to provide minimum A_v:

$$s = \frac{A_v f_y}{0.75 \times \sqrt{f'_c}\, b_w} = \frac{(0.4)(60{,}000)}{0.75 \times \sqrt{4{,}000}\,(18)} = 28.1 \text{ in.}$$

$$s = \frac{A_v f_y}{50\, b_w} = \frac{(0.4)(60{,}000)}{50(18)} = 26.7 \text{ in.}$$

$$4\sqrt{f'_c}\, b_w d = \frac{4\sqrt{4{,}000}\,(18)(21.5)}{1{,}000} = 97.9 \text{ kips}$$

$$V_s = \frac{104.0 - (2)(18.4)}{0.75} = 89.6 \text{ kips} < 97.9 \text{ kips}$$

$$\therefore \text{Max } s = \frac{d}{2} = \frac{21.5}{2} = 10.75 \text{ in.}$$

$\therefore$ Use #4 stirrups @ 10 in. o.c.

ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Solution 604, p. 113

The fourth line from the bottom should read as follows:

$$Z_\theta' = C_d C_g Z_\theta = (0.9)(0.95)(3{,}336) = 2{,}852 \text{ lb}$$

Solution 703, pp. 123–128

Requirement (a), p. 123, lines 8–20, should read as follows:

$W_n = 16.5 - (4)(15/16) + (2)(3.5)^2/[(4)(3)]$
$\quad = 14.8$ in.
$A_n = (14.8)(1.26)$
$\quad = 18.65 \text{ in}^2$
$A_g = b_f \times t_f$
$\quad = (16.5)(1.26)$
$\quad = 20.79 \text{ in}^2$
$\phi_u = 0.80$ Art. 6.5.4.2
$\phi_y = 0.95$
$F_u = 58$ Table 6.4.1-1
$F_{yt} = 36$

$$A_e = \frac{(0.8)(58)}{(0.95)(36)}\left(18.65 \text{ in}^2\right)$$

$$= (1.357)\left(18.65 \text{ in}^2\right) = 25.31 \text{ in}^2 > A_g = 20.79 \text{ in}^2 \text{ Governs.}$$

Requirement (a), p. 124, line 26, should read as follows:

$R_h = 1.0$ rolled shape

Requirement (b), p. 127, lines 15–16, should read as follows:

$L_c = 2 - 0.875$-in.-dia./2 Clear distance from edge of hole to edge of connected plate
$\quad = 1.56$ in. $< 2d = 1.75$ in.

ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Requirement (b), p. 128, lines 1–6, should read as follows:

Bearing on splice plates:

ϕR_n = (0.8)(1.2)(1.56)(5/8)(58) where ϕ_{bb} = 0.8

= 54.3 kips per bolt Does not control bolt strength

Bearing on beam flange:

ϕR_n = (0.8)(1.2)(1.56)(1.26)(58)

= 109.4 kips per bolt Does not control bolt strength

Requirement (b), p. 128, line 31, should read as follows:

R_r = R_n = (411.84) = 411.8 kips > 255.3 kips OK Eq. 6.13.2.2-1

Question 801, p. 180

Requirement (b) should read as follows:

(b) Determine the design wind pressure and seismic design force on the parapet. For wind, use the provisions of ASCE 7 Ch. 30 Part 6 and neglect corner zones. (Consider interior zones only.)

Solution 134, p. 216

The last two NDS tables referenced at the end of the solution should be 11.3.1.

$K_F = 3.32$ NDS Table 11.3.1

$\phi_Z = 0.65$ NDS Table 11.3.1

ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Solution 803, p. 230
The last five lines of Requirement (c) should read as follows:

Alternatively, the provisions of Sec. 25.4.2.3 may be used

$$\ell_d = \frac{3}{40}\frac{f_y\ \psi_t\psi_e\psi_s}{\lambda f_c'\left(\frac{c_b+K_{tr}}{d_b}\right)}d_b \qquad \text{ACI Eq. 25.4.2.3a}$$

where $\frac{c_b+K_{tr}}{d}=\frac{3.313+0}{0.625}=5.3\le 2.5$ Use 2.5

$$\ell_d=\frac{3}{40}\frac{60(1.0)(1.0)(0.8)}{1.0(5)(2.5)}0.625=12.7\text{ in.}$$

Class B splice $=(1.3)(12.7)=16.5$ NG

ERRATA for
Structural Engineering Sample Questions and Solutions
ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Solution 804, p. 232
Requirement (b) should read as follows:

(b) Nailing requirements of shear wall: NDS SDPWS Table 4.3A

15/32" wood structural panels-sheathing
w/ 8d nails @ 6" o.c. @ panel edges
and @ 12" o.c. @ intermediate supports, V_w = 730 plf
Footnote 3 specific adjustment factor:
= [1 – (0.5 – G)]
Hem-Fir G = 0.43 NDS Table 12.3.3A
= [1 – (0.5 – 0.43)] = 0.93

$$V_{Allow} = \frac{730 \text{ plf}}{2.0} \times 0.93 = 340 \text{ plf} > 270 \text{ plf} \quad \text{OK}$$

Bottom plate to blocking between trusses NDS Table 12N
For 16d nails and 2×4 bottom plate (t_s = 1 1/2")
Z = 122 lb
Penetration into main member (blocking):

$$p = 3\,1/2 - 1\,1/2 - 3/4 = 1\,1/4"$$

$$6\text{ D} = 6\,(0.162) = 0.972"$$

$$10\text{ D} = 10(0.162) = 1.62"$$

$$\therefore 6\text{ D} < p < 10\text{ d} \rightarrow \text{use adj. factor footnote 3}$$

$$z' = 122\text{ lb} \times C_D \times p / 10\text{ d}$$

$$= 122 \times 1.6 \times 1.25 / 1.62 = 150\text{ lb / nail}$$

$$\text{Required spacing} = \frac{150}{270} = 0.56' = 6.7"$$

$$\therefore \text{Attach bottom plate to blocking with 16d nails @ 6" o.c. (max.)}$$

Add second-floor diaphragm loads

$$V_{DIA} = \frac{3{,}130 \text{ lb}}{30 \text{ ft}} = 104.3 \text{ plf}$$

V = 270 plf + 105 plf

V = 375 plf

ISBN 978-1-932613-89-6

804. (Continued)

Blocking between trusses to top plate (wall below)

Use 16d toe nails — NDS Table 12N

z = 122 lb (from above)

Penetration of toe nail into main member (top plate):

$$p = \ell \cos 30° - \ell/3 = 31/2\,(\cos 30°) - \frac{31/2}{3} = 1.86"$$

$$\therefore p > 10\,d$$

$$z' = 122\text{ lb} \times C_D \times C_{tn}$$

$$= 122 \times 1.6 \times 0.83 = 162\text{ lb / nail}$$

$$\text{Required spacing} = \frac{162}{375\text{ plf}} = 0.43\text{ ft} = 5.18\text{ in.}$$

$\therefore$ Attach blocking to top plate with 16d toe nails @ 5 in. o.c. max.

Alternately, provide metal framing clips from blocking to top plate with correct combination of capacity and spacing for overall resistance of 285 plf

Net uplift holdown forces:

At location adjacent to balcony:

M_{gross} = 56,425 ft-lb (from Requirement (a))

$$M_{0.6\,D} = 0.6(20\text{ psf})(20\text{ ft})(10\text{ ft})(10\text{ ft}/2)$$

$$+\,0.6(15\text{ psf})(20\text{ ft})(10\text{ ft})(10\text{ ft}/2) = 21{,}000\text{ ft-lb}$$

$$M_{net} = 56{,}425 - 21{,}000 = 35{,}425\text{ ft-lb}$$

$$\text{Distance between holdown bolts} \approx 10\text{ ft} - 0.75\text{ ft} = 9.25\text{ ft}$$

$$T_{@holdown} = \frac{M}{b} = \frac{35{,}425\text{ ft-lb}}{9.25\text{ ft}} = 3{,}830\text{ lb}$$

At location adjacent to 10-ft opening:

$$T_{@\,holdown} = T_{shear\ wall} + T_{header}$$

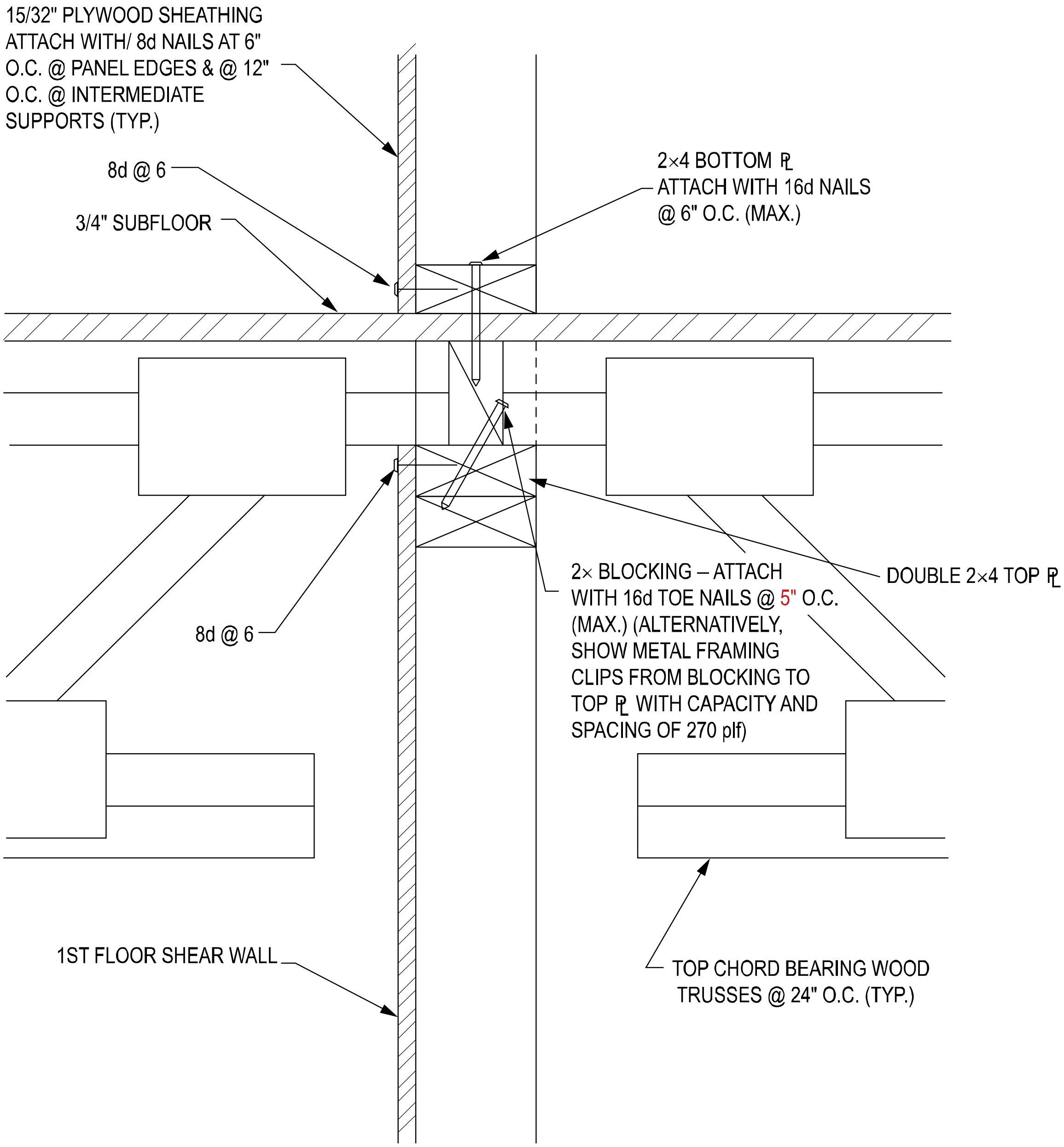
$$T_{header} = 440\text{ plf }(10\text{ ft}/2) - 0.6(20\text{ psf} + 15\text{ psf})(20\text{ ft})\,(10\text{ ft}/2) = 100\text{ lb}$$

$$\therefore\ T_{@\,holdown} = 3{,}830 + 100 = 3{,}930\text{ lb}$$

ERRATA for
Structural Engineering Sample Questions and Solutions
ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Solution 804, p. 235
Requirement (d) should read as follows:



Note: This detail outlines one of numerous possible configurations. The key components for the load path include:

1. Plywood wall sheathing
2. Boundary nailing
3. Bottom plate
4. Nailing of plate to plywood floor sheathing and blocking
5. Nailing of blocking to double top plate
6. Boundary nailing
7. Plywood wall sheathing

ISBN 978-1-932613-89-6

Errata posted 4/29/2019

Solution 901, p. 239
Line 12 should read as follows:

$$A_{v_{min}} \geq 0.0316\sqrt{f'_c}\,\frac{b_v s}{f_y} = 0.0316\sqrt{3.5}\,\frac{48.0(6)}{60} = 0.28\ \text{in}^2 \qquad \text{OK}$$