**ERRATA for**
***Structural Engineering Practice Exam***
ISBN 978-1-932613-89-6

Errata posted 7-12-2018

**Revisions are shown in red.**

## Vertical Forces AM

### 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 |

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## Vertical Buildings PM

### 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.**

## Vertical Forces AM

### Solution 124, p. 83

**LRFD option:**

$$M_n = S_e F_y \qquad \text{Eq. C3.1.1-1}$$

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

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

**THE CORRECT ANSWER IS: (B)**

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**Solution 134, p. 89**

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

$M_{max} = F_b bkjd^2/[2(12)] = 900(12)(0.207)(0.931)(7.625/2)^2/[2(12)]$

$= 1{,}260$ ft-lb/ft

**Vertical Buildings PM**

**Solution 601, Requirement c, p. 101**

**(c) Wall-to-footing connection**

Line 15 should read as follows:

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

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**Solution 603, Requirement c, 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 $A_{s\,provided} = 3.95\text{ in}^2$

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**603. (Continued)**

Negative moment at exterior face of first interior column:

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

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

From design aids table, $\rho = 0.0158$

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

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

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

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

Shear:

$V_u = 104.0$ kips

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

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

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

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.

*Previously posted errata continued on next page*

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Revisions are shown in red.

**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 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
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**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

$$V_{DIA} = \frac{3{,}130\text{ lb}}{7 \times 30\text{ ft}} = 14.9\text{ plf}$$

V = 270 plf + 15 plf

V = 285 plf

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**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):

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

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

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

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

$$\text{Required spacing} = \frac{162}{285\text{ plf}} = 0.57\text{ ft} = 6.82\text{ in.}$$

$\therefore$ Attach blocking to top plate with 16d toe nails @ 6 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 psf)(20 ft)(10 ft)(10 ft/2)

+ 0.6(15 psf)(20 ft)(10 ft)(10 ft/2) = 21,000 ft-lb

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

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

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

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

At location adjacent to 10-ft opening:

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

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

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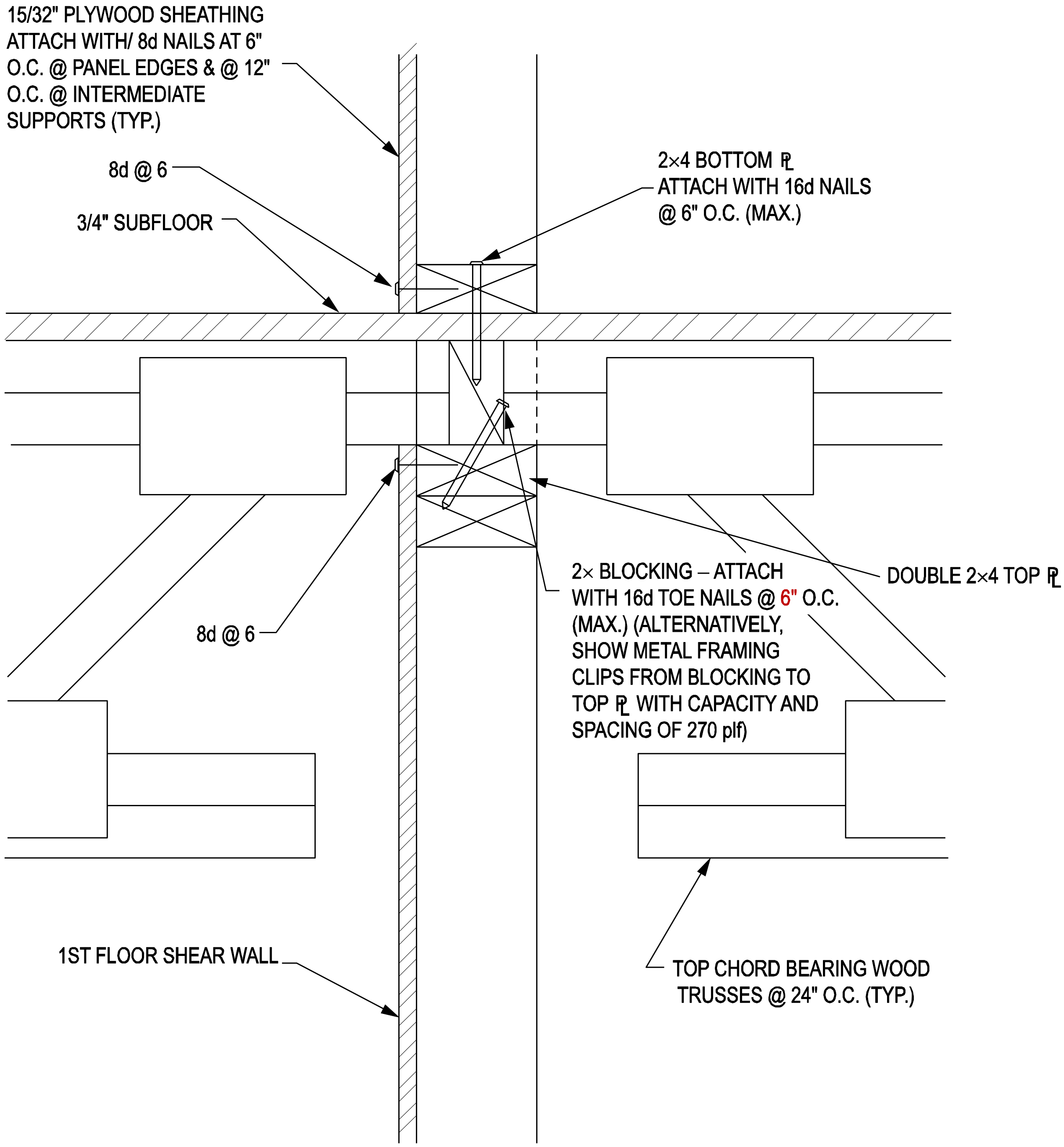
## Solution 804, p. 235

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