Unified Design of Steel Structures, 3rd Edition, Errata **last updated 2/13/22**

p. 78, Section 3.8, Problem 2. *"*the 2010 AISC *Specification*" should be "the 2016 AISC *Specification"*

p. 112, Section 4.8, see p.3 of this errata

p. 121, Section 4.13, Problem 28. The end of the WT (perpendicular to the load) is also welded.

p. 184, Section 5.12, Problem 32. "A300 Gr. C" should be "A500 Gr. C"

p. 215, Section 6.4.2, Example 6.7

Example 6.7a

Step 6: Select a shape with $\phi M_p \geq 282$ ft-kips from *Manual* Table 3-2.

Thus , select

$$\text{W18×40}, \ \phi M_p = 294 \text{ ft-kips}$$

Step 7: Confirm in *Manual* Table 3-10 that the line, either solid or dashed, for the W18×40 is above and to the right of the intersection of $\phi M_n = 169$ ft-kips and L_b = 15 ft. Since this is not the case, ~~Thus, select~~ the W18×40 is not adequate.

Step 8: By trial and error, checking the shapes that are up and to the right of this point in Table 3-10, select

W16×45

with $\phi M_p = 309 \geq 282$ ft-kips

Example 6.7b

Step 6: Select a shape with $M_p/\Omega \geq 188$ ft-kips from *Manual* Table 3-2.

Thus, select a

$$\text{W18×40 with } M_p/\Omega = 196 \text{ ft-kips}$$

Step 7: Confirm in *Manual* Table 3-10 that the line, either solid or dashed, for the W18×40 is above and to the right of the intersection of $M_n/\Omega = 113$ ft-kips and L_b = 15 ft. Since this is not the case, ~~Thus, select~~ the W18×40 is not adequate.

Step 8: By trial and error, checking the shapes that are up and to the right of this point in Table 3-10, select

W16×45

with $M_p/\Omega = 205 \geq 188$ ft-kips

p. 262, Section 6.17, Problem 48. "limit deflection" should be "limit live load deflection"

p. 262, Section 6.17, Problem 55. Use 30 ft spans.

p. 272, Section 7.2.2, Flange Local Buckling. "...same as those used in ~~Chapter 6 and~~ Section 7.2.1."

p. 275, Section 7.2.2, Example 7.1. In Step 2, insert

$$\lambda_w = \frac{h_c}{t_w} = \frac{48}{0.375} = 128 > \lambda_{pw} = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29000}{36}} = 107$$

$$128 < \lambda_{rw} = 5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{29000}{36}} = 162$$

before "Thus, this is a noncompact web girder, and the provisions of Section F4 must be followed. The web plastification factor must be determined."

p. 332, "EXAMPLE 8.3a" should be "EXAMPLE 8.3b"

p. 333, in the ASD solution for Example 8.3 (changes in red font)

Step 5: Check the W14×132 for combined axial load and bending. To determine which equation to use, check

$$\frac{P_n}{\Omega} = \frac{530}{960} = 0.552 \geq 0.2$$

p. 343, Example 8.6a, Step 7 – for the sway amplification, “P_u” should be replaced with “P_{story}”

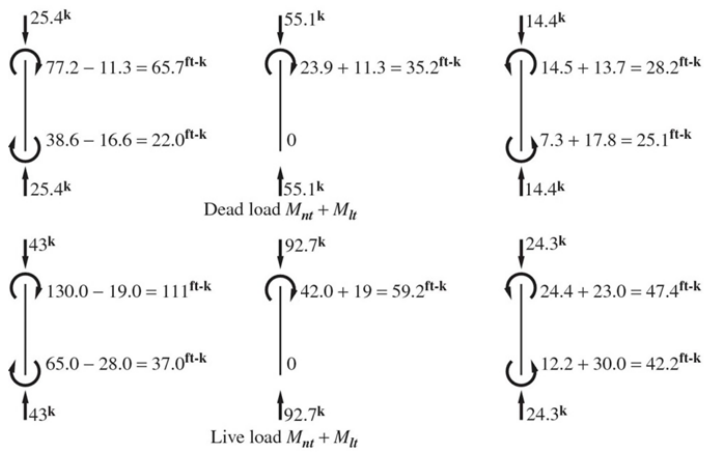
p. 367, Section 8.14, Problem 14. The total story load, P_{story}, is 15 times the individual column load; “an moment frame” should be “a moment frame”

p. 367, Section 8.14, Problem 16. Determine by (a) LRFD and (b) ASD.

p. 368, Section 8.14, Problem 18. “0.5 in.” should be “0.1 in.”

p. 368, Section 8.14, Problem 23. “an LRFD second-order direct analysis” should be “an ASD second-order direct analysis”

p. 369, Figure P8.19. Interior column moments are acting in the same direction; see revised figure:



p. 370, Figure P8.28. For the loads given at the top of the figure, w_{LL} is 1.7 k/ft and w_{DL} is 2.8 k/ft.

p. 379, third paragraph, If $C_c < T_w + T_f$, less tension is needed for equilibrium and the PNA ~~is~~ MAY BE in the web.

p. 386, Example 9.3, Step 6. $M_n = T_s\left(\frac{d}{2}\right) + C_c\left(t - \frac{a}{2}\right) - 2A_{s-c}F_y\left(\frac{x}{2}\right)$

p. 415, Example 9.9, Step 3, EI_{eff} equation should show C_1 rather than C_3

p. 426, Section 9.13, Problems. If needed, assume normal-weight concrete.

p. 446, Example 10.3, Step 1 should read $F_{nv} = 0.45F_u = 0.45(150) = 68$ ksi

p. 468, Figure P10.28. Workable gage should be 3-1/2 in. instead of 5-1/2 in. Does not change problem.

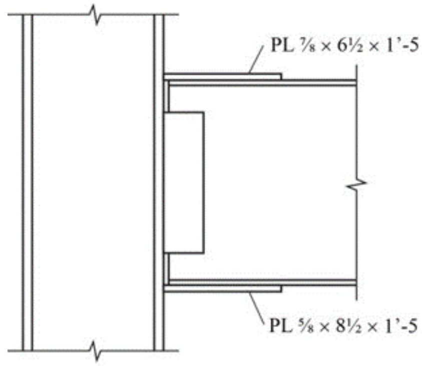
p. 503, Example 11.4a, Step 12. $\phi R_n = 55.8$ kips

p. 544, Section 11.12, Problem 18. Should have a dead load reaction of 7.5 kips and a live load reaction of 22.5 kips

p. 545, Section 11.12, Problem 41. Assume $f'_c = 5$ ksi; assume also that the area of the concrete support is the same as the steel bearing plate area.

p. 545, Section 11.12, Problem 42. Assume $f'_c = 5$ ksi; assume also that the area of the concrete support is 4 times as large as the steel bearing plate area, and assume concentric areas.

p. 556, Figure 12.4(c), below, should be inserted.



(c) Connection geometry

pp. 569-570, 575-576, Examples 12.3a and 12.3b, see below and pp. 4-6 of this errata

pp. 582-583, 586, Section 12.4, Examples 12.4a and 12.4b. Step 1, between the two equations, the limit state should be "flange local bending"; Steps 2 and 3, F_{yf} should be F_{yw}

p. 585, Figure 12.8, $w = b_s$

Section 4.8 PIN-CONNECTED MEMBERS (starting on p. 112)

The discussion in this section does not mention the two arbitrary requirements of Section D5.2(c).

> The width of the plate at the pin hole shall not be less that $2b_e + d$ and the minimum extension, a, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than $1.33b_e$.

Since b_e is limited to the width of the plate, the first of these will automatically be satisfied. The second, however must be checked.

In Example 4.15a and 4.15b, the end distance is taken as 2.5 in. but this is less than $1.33b_e = 1.33(2.13) = 2.83$ in. Therefore the end distance should be taken as $a = 3.0$ in. Thus, in Step 4, the shear area will be

$$A_{sf} = 2t(a + d/2) = 2(0.750)(3.00 + 4.0/2) = 7.50 \text{ in.}^2$$

The nominal strength for the limit state of shear rupture using Equation D5-2 will be

$$P_n = 0.6F_u A_{sf} = 0.6(65)(7.50) = 293 \text{ kips}$$

and the available strength will be

$$\phi P_n = 0.75(293) = 220 \text{ kips}$$
$$P_n/\Omega = 293/2.00 = 147 \text{ kips}$$

Note that the change in the extension past the pin has not changed the strength of the member since strength is controlled by the limit state of tension rupture in Step 3.

Example 12.3a errata. Pages 569-570 (changes in red font)

Step 8: Check the plate for block shear rupture.

Check the plate for block shear using the geometry shown in Figure 12.5. There are three possible block shear failure patterns shown, one with the center portion failing in tension, Figure 12.5a, one with the two outside portions failing in tension, Figure 12.5b, and one with only one shear failure plane and the tension failure over the middle portion with one outside portion, Figure 12.5c. The worst case must be identified. For the first two possibilities, the critical tension area for block shear will be the one associated with the least tension width. In this case it will be for the middle 3-1/2 in. section as shown in Figure 12.5a, and the critical net tension area is

$$\begin{aligned} A_{nt} &= \left(b - \left(d_h + 1/16\right)\right)t_p \\ &= \left(3.5 - \left(7/8 + 1/8\right)\right)(0.750) = 1.88 \text{ in.}^2 \end{aligned}$$

and the shear areas for the two shear planes are

$$\begin{aligned} A_{gv} &= 2lt_w \\ &= 2\left(10.75(0.750)\right) = 16.1 \text{ in.}^2 \\ A_{nv} &= 2\left(l - \left(n - 0.5\right)\left(d_h + 1/16\right)\right)t_w \\ &= 2\left(10.75 - 3.5\left(7/8 + 1/8\right)\right)(0.750) = 10.9 \text{ in.}^2 \end{aligned}$$

Determine the tension rupture strength

$$F_u A_{nt} = 58\left(1.88\right) = 109 \text{ kips}$$

Consider the shear yield and shear rupture and select the one with least strength; thus,

$$\begin{aligned} 0.6F_y A_{gv} &= 0.6\left(36\right)\left(16.1\right) = 348 \text{ kips} \\ 0.6F_u A_{nv} &= 0.6\left(58\right)\left(10.9\right) = 379 \text{ kips} \end{aligned}$$

Selecting the shear yield term and combining it with the tension rupture term gives a connection design block shear strength, with U_{bs} = 1.0, of

$$\phi R_n = 0.75\left(348 + 1.0\left(109\right)\right) = 0.75\left(457\right) = 343 > 167 \text{ kips}$$

Since the shear contribution is so much greater than the tension contribution, the pattern shown in Figure 12.5c should be checked. For a tension width equal to 5.38 in., the net tension area is

$$\begin{aligned} A_{nt} &= \left(b - 1.5\left(d_h + 1/16\right)\right)t_p \\ &= \left(5.38 - 1.5\left(7/8 + 1/8\right)\right)(0.750) = 2.91 \text{ in.}^2 \end{aligned}$$

and the shear areas for a single shear plane are

$$
\begin{aligned}
A_{gv} &= lt_w \\
&= 10.75(0.750) = 8.06 \text{ in.}^2 \\
A_{nv} &= \left(l - (n - 0.5)(d_h + 1/16)\right)t_w \\
&= \left(10.75 - 3.5(7/8 + 1/8)\right)(0.750) = 5.44 \text{ in.}^2
\end{aligned}
$$

Determine the tension rupture strength

$$F_u A_{nt} = 58(2.91) = 169 \text{ kips}$$

Consider shear yield and shear rupture and select the one with the least strength; thus

$$0.6F_y A_{gv} = 0.6(36)(8.06) = 174 \text{ kips}$$

$$0.6F_u A_{nv} = 0.6(58)5.44 = 189 \text{ kips}$$

Selecting the shear yield term and combining it with the tension rupture term gives a connection design block shear strength, with U_{bs} = 1.0, of

$$\phi R_n = 0.75\left(174 + 1.0(169)\right) = 0.75(343) = 257 > 167 \text{ kips}$$

Of the three block shear patterns illustrated, this is clearly the most critical. However, it should be noted from step 4 that tension rupture of the plate is more critical than any of the block shear failure patterns. When the shear strength of the block shear patterns is larger than all the potential tension pattern strengths, tension rupture of the plate will likely control over any of the potential block shear modes. Thus, any time that the pattern illustrated in Figure 12.5c would be the controlling block shear pattern, tension rupture of the plate will be more critical.

Example 12.3b errata. Pages 575-576 (changes in red font)

Step 8: Check the plate for block shear rupture.

Check the plate for block shear using the geometry shown in Figure 12.5. There are three possible block shear failure patterns shown, one with the center portion failing in tension, Figure 12.5a, one with the two outside portions failing in tension, Figure 12.5b, and one with only one shear failure plane and the tension failure over the middle portion with one outside portion, Figure 12.5c. The worst case must be identified. For the first two possibilities, the critical tension area for block shear will be the one associated with the least tension width. In this case it will be for the middle 3½ in. section as shown in Figure 12.5a, and the critical net tension area is

$$
\begin{aligned}
A_{nt} &= \left(b - (d_h + 1/16)\right)t_p \\
&= \left(3.5 - (7/8 + 1/8)\right)(0.750) = 1.88 \text{ in.}^2
\end{aligned}
$$

and the shear areas for the two shear planes are

$$A_{gv} = 2lt_w$$
$$= 2\left(10.75(0.750)\right) = 16.1 \text{ in.}^2$$
$$A_{nv} = 2\left(l - (n-0.5)(d_h + 1/16)\right)t_w$$
$$= 2\left(10.75 - 3.5(7/8 + 1/8)\right)(0.750) = 10.9 \text{ in.}^2$$

Determine the tension rupture strength

$$F_u A_{nt} = 58(1.88) = 109 \text{ kips}$$

Consider the shear yield and shear rupture and select the one with least strength; thus,

$$0.6F_y A_{gv} = 0.6(36)(16.1) = 348 \text{ kips}$$
$$0.6F_u A_{nv} = 0.6(58)(10.9) = 379 \text{ kips}$$

Selecting the shear yield term and combining it with the tension rupture term gives a connection allowable block shear strength, with U_{bs} = 1.0, of

$$R_n/\Omega = \left(348 + 1.0(109)\right)/2.00 = 457/2.00 = 229 > 111 \text{ kips}$$

Since the shear contribution is so much greater than the tension contribution, the pattern shown in Figure 12.5c should be checked. For a tension width equal to 5.38 in., the net tension area is

$$A_{nt} = \left(b - 1.5(d_h + 1/16)\right)t_p$$
$$= \left(5.38 - 1.5(7/8 + 1/8)\right)(0.750) = 2.91 \text{ in.}^2$$

and the shear areas for a single shear plane are

$$A_{gv} = lt_w$$
$$= 10.75(0.750) = 8.06 \text{ in.}^2$$
$$A_{nv} = \left(l - (n-0.5)(d_h + 1/16)\right)t_w$$
$$= \left(10.75 - 3.5(7/8 + 1/8)\right)(0.750) = 5.44 \text{ in.}^2$$

Determine the tension rupture strength

$$F_u A_{nt} = 58(2.91) = 169 \text{ kips}$$

Consider shear yield and shear rupture and select the one with the least strength; thus

$$0.6F_y A_{gv} = 0.6(36)(8.06) = 174 \text{ kips}$$
$$0.6F_u A_{nv} = 0.6(58)5.44 = 189 \text{ kips}$$

Selecting the shear yield term and combining it with the tension rupture term gives a connection allowable block shear strength, with U_{bs} = 1.0, of

$$R_n/\Omega = \left(174 + 1.0(169)\right)/2.00 = 343/2.00 = 172 > 111 \text{ kips}$$

Of the three block shear patterns illustrated, this is clearly the most critical. However, it should be noted from step 4 that tension rupture of the plate is more critical than any of the block shear failure patterns. When the shear strength of the block shear patterns is larger than all the potential tension pattern strengths, tension rupture of the plate will likely control over any of the potential block shear modes. Thus, any time that the pattern illustrated in Figure 12.5c would be the controlling block shear pattern, tension rupture of the plate will be more critical.