

CISC HANDBOOK OF STEEL CONSTRUCTION

12th Edition, 1st Printing, May 2021

REVISION LIST - 6 DECEMBER 2021

The following revisions to the First Printing of the CISC *Handbook of Steel Construction*, 12th Edition, will be incorporated into future printings. Minor editorial revisions are not shown.

Page(s)	Revisions
3-90 to 3-91	<i>Replace pages 3-90 and 3-91 with the following pages. Revisions are highlighted in red.</i>

Solution:

(a) Web Connection

The design of the connection between the beam web and the column flange need only account for the vertical shear, neglecting eccentricity. (Design for 130 kN shear.)

Combined shear and moment does not govern for the beam (S16:19 Clause 14.6, calculation not shown).

Two alternatives are shown to illustrate a field-welded and a field-bolted condition.

Alternative 1

Single plate field-welded to the beam web, shop-welded to the column flange, holes for two $\frac{3}{4}$ in. erection bolts

To resist the factored shear, try 5 mm fillet welds ($X_u = 490$ MPa) on a 6 mm plate.

The required weld length is $130/0.778 = 167$ mm (Table 3-24(a))

Try a 230 mm long plate, for a W410 beam (Table 3-38)

Check the plate for the factored shear capacity. (Clause 13.4.3)

Gross plate area: $A = 230 \times 6 = 1380 \text{ mm}^2$

$$V_r = \phi 0.66 F_y A = 0.9 \times 0.66 \times 300 \times 1380 / 1000 = 246 \text{ kN} > 130 \text{ kN}$$

Use a $6 \times 75 \times 230$ plate and 5 mm fillet welds with $X_u = 490$ MPa (matching condition).

Alternative 2

Single plate shop-welded to the column flange, field-bolted to the beam web with $\frac{7}{8}$ in. ASTM F3125 grade A325 bolts (or $\frac{3}{4}$ in. bolts if also used to connect the flange plates)

From Table 3-4, factored shear resistance, single shear, threads intercepted, for $\frac{7}{8}$ in. A325 bolts = 108 kN per bolt:

$$\text{For 2 bolts, } V_r = 2 \times 108 = 216 \text{ kN} > 130 \text{ kN}$$

Check the factored bearing resistance on the beam web, $w = 7.7$ mm

From Table 3-6(a), B_r for $t = 7$ mm is 168 kN per bolt > 79.0 kN

Try a 6 mm plate, 230 mm long, 2 bolts at 160 mm pitch:

Bearing resistance on a 6 mm plate, Table 3-6(b), $B_r = 141$ kN per bolt > 79.0 kN

Required plate thickness (based on shear resistance, Clause 13.11) is:

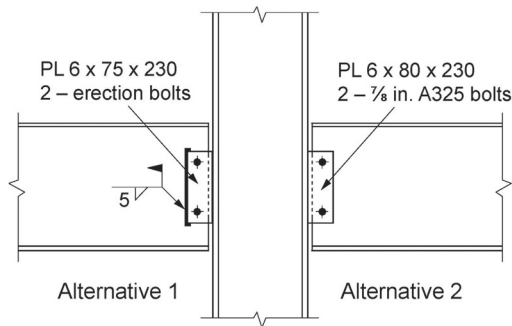
$$130 \times 10^3 / [0.75 \times 0.6 \times 230 (300 + 440)/2] = 3.4 \text{ mm} < 6 \text{ mm}$$

Block shear resistance (tension + shear, Clause 13.11) is adequate (not shown).

Use a $6 \times 80 \times 230$ plate and two $\frac{7}{8}$ in. A325 bolts at 160 mm pitch.

(b) Flange Connection

Two alternatives are shown to illustrate field-bolted and field-welded conditions.



Alternative 2 replaces the two erection bolts with permanent high-strength bolts, and eliminates vertical field welding (likely a better solution).

Alternative 1

Top and bottom moment plates shop-welded to the column, field-bolted to beam flanges with A325 bolts

The flange force due to factored loads is $310 \times 1000 / 407 = 762 \text{ kN}$

Bolts

Assuming a joint length, $L < 760 \text{ mm}$, from Table 3-4, the required number of $\frac{7}{8}$ in. A325 bolts (threads excluded) is:

$$762 / 154 = 4.95 \quad \text{Use 6 bolts (2 rows of 3; } L = 2(80) = 160 < 760 \text{ mm)}$$

$$\text{Shear per bolt} = 762 / 6 = 127 \text{ kN}$$

Beam

Moment resistance at sections with bolt holes in the flanges (Clauses 14.1.2 and 14.1.3):

When the flange area reduction exceeds 15% of the gross area and the holes in the compression flange are filled with $\frac{7}{8}$ -inch bolts, the table for *Beams with Flange Holes* in Part 5 can be used (assuming two 1-inch bolts in the tension flange, conservatively).

$$Z_{xe} = 1080 \times 10^3 \text{ mm}^3, \quad M_r = \phi Z_{xe} F_y = 335 \text{ kN}\cdot\text{m} > M_f = 310 \text{ kN}\cdot\text{m}.$$

Other design checks may apply (e.g., Clause 13).

Factored bearing resistance, $t = 12.8 \text{ mm}$

From Table 3-6(a), for $t = 12 \text{ mm}$, $B_r = 288 \text{ kN per bolt} > 127 \text{ kN}$

Block Shear (Cl. 13.11). Try 80 mm pitch, 70 mm end distance and 35 mm edge distance.

(i) Edge block shear pattern, Figure 3-4(a)

$$\begin{aligned} T_r &= 0.75 (12.8) [1.0(2 \times 35 - 26)450 + 0.6(2)(70 + 2 \times 80)(345 + 450)/2] / 1000 \\ &= 1240 \text{ kN} > 762 \text{ kN} \end{aligned}$$

(ii) Tear-out pattern, Figure 3-4(b): not critical (calculation not shown)

Flange Plates

Required gross area = $762 \times 10^3 / (0.9 \times 300) = 2820 \text{ mm}^2$ (Clause 13.2(a), tension)

Required net effective area = $762 \times 10^3 / (0.75 \times 440) = 2310 \text{ mm}^2$

Try a 200×16 plate, and check areas.