CISC HANDBOOK OF STEEL CONSTRUCTION

12th Edition, 1st Printing, May 2021

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The following revisions to the 1st Printing of the CISC *Handbook of Steel Construction,* 12th Edition, will be incorporated in future printings. The list includes all previously published errata for the 1st Printing.

Revisions are highlighted in red. Minor editorial revisions are not shown.

Page(s)	**Revisions**
3-20 to 3-21	*Replace pages 3-20 and 3-21 with the following pages.*

BOLTS IN TENSION AND PRYING ACTION

General

Connections with fasteners loaded in tension occur in many common situations, such as hanger and bracing connections with tee-type gussets, and end-plate moment connections. When bolts are loaded in direct tension, Clause 13.12.1.3 of CSA S16:19 requires that the effects of prying action be taken into account in proportioning the bolts and connected parts. This clause also requires that the connection be arranged to minimize prying forces when subjected to tensile cyclic loading.

The actual stress distribution in the flange of a tee-type connection is extremely complex as it depends on the bolt size and arrangement, and on the strength and dimensions of the connecting flange. Consequently, various design methods have been proposed in the technical literature for proportioning such connections. The procedures given in this section are based on the recommendations contained in the *Guide to Design Criteria for Bolted and Riveted Joints,* by Kulak, Fisher and Struik, second edition, page 285.

The procedures include a set of seven equations for selecting a trial section and for evaluating the bolt forces and flange capacity. Equation (4) uses the full tensile resistance, T_r, of the bolts to determine α for use in equation (5) which provides the maximum connection capacity. Similarly, equation (6) uses the applied factored tensile load per bolt, P_f, to determine α for use in the amplified bolt force expressed by equation (7). This provides a value for the factored load per bolt (including prying), T_f.

Based on these equations, Table 3-13 and Figure 3-1 provide aids for preliminary design and checking purposes. They indicate the effect of applied factored tensile load per bolt and flange geometry for various bolt sizes, assuming static loads.

In general, prying effects can be minimized by dimensioning for minimum practical gauge distance and for maximum permissible edge distance. For repeated loading the flange must be made sufficiently thick and stiff so that flange deformation is virtually eliminated. In addition, special attention must be paid to bolt installation to ensure that the bolts are properly pretensioned to provide the required clamping force.

Paragraph, equations and figure deleted

Equations

$$K = \frac{4\times10^3\, b'}{\phi\, p\, F_y} \qquad (1)$$

$$\delta = 1 - \frac{d'}{p} \qquad (2)$$

$$\text{Range of } t = \sqrt{\frac{K P_f}{1+\delta\alpha}} \qquad (3)$$

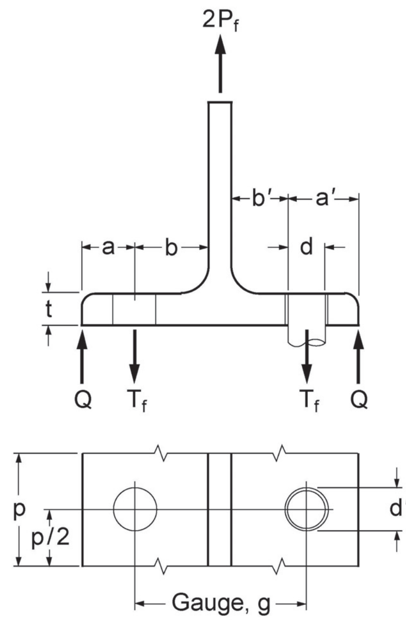
t_{min} when $\alpha = 1.0$, t_{max} when $\alpha = 0.0$

$$\alpha = \left(\frac{K T_r}{t^2} - 1\right)\frac{a'}{\delta(a'+b')}, \quad 0 \le \alpha \le 1.0 \qquad (4)$$

$$\text{Connection capacity } = \frac{t^2}{K}(1+\delta\alpha)n \qquad (5)$$

$$\alpha = \left(\frac{K P_f}{t^2} - 1\right)\frac{1}{\delta} \quad \text{(for use in Eq. 7)} \qquad (6)$$

$$T_f \approx P_f\left[1 + \frac{b'}{a'}\left(\frac{\delta\alpha}{1+\delta\alpha}\right)\right] \le T_r \qquad (7)$$



Nomenclature

K	=	Parameter as defined in Eq. 1
P_f	=	Applied factored tensile load per bolt, (kN)
Q	=	Prying force per bolt at factored load, $Q = T_f - P_f$, (kN)
T_f	=	Factored load per bolt including prying (amplified bolt force), (kN)
T_r	=	Factored tensile resistance per bolt, $0.75\phi_b A_b F_u$, (kN)
F_y	=	Yield strength of flange material, (MPa)
a	=	Distance from bolt line to edge of tee flange, not more than $1.25\,b$, (mm)
a'	=	$a + d/2$, (mm)
b	=	Distance from bolt line (gauge line) to face of tee stem, (mm)
b'	=	$b - d/2$, (mm)
		For double-angle connections, b and b' are measured from the middle line of the vertical angle leg.
d	=	Bolt diameter, (mm)
d'	=	Nominal hole diameter, (mm).
n	=	Number of flange bolts in tension
p	=	Length of flange tributary to each bolt, or bolt pitch, (mm)
t	=	Thickness of flange, (mm)
α	=	Ratio of sagging moment at bolt line to hogging moment at stem of tee
δ	=	Ratio of net to gross flange area along a longitudinal line of bolts (see Eq. 2)
ϕ	=	Resistance factor for the tee material, (0.9)

Page(s)	**Revisions**
3-84	*Replace page 3-84 with the following page. The "Solution" of the design example has been moved to page 3-85.*

STIFFENED SEATED BEAM CONNECTIONS

Table 3-45 lists factored resistances of stiffened seats for the tee-shaped weld configuration shown. Capacities are based on the use of matching electrodes, X_u = 490 MPa, and steel with F_y = 300 MPa. They may be used conservatively for steel with F_y = 345 MPa or 350 MPa. Factored resistances are computed using the formulas given in the section, *Eccentric Loads on Weld Groups, Shear and Moment*. For seats having a thin and narrow stiffener, yielding or crippling resistance of the stiffener as determined in accordance with S16:19 Clause 14.3.2(b) governs the tabulated values.

The figures in Table 3-45 show the general arrangement. Although the horizontal welds connecting the seat plate to the support were ignored in the calculations, they are provided for stability. Welds smaller than the vertical stiffener welds may be used, and they do not intersect the vertical welds. Generally, the seat plate is connected to the stiffener with welds having a minimum shear resistance equal to the capacity of the welds connecting the seat plate to the supporting member. Welds or bolts may be used to connect the supported beam to the seat and to attach the clip angle required to stabilize the beam. If welds are used, the seat should be long enough to accommodate the fillet welds as shown in the figure. If bolts are used, the seat length should match or exceed the flange width of the beam.

Stiffened seats must be proportioned so that the stiffener thickness t is not less than the web thickness w of the supported beam (for beams with unstiffened webs). If the beam has a higher specified yield strength than the stiffener, the relationship, $t \times F_y$ (stiffener) = $w \times F_y$ (beam) shall be satisfied.

When stiffened seats are in line on opposite sides of a column web, the size of the vertical fillet welds (with X_u = 490 MPa) shall not exceed $F_y / 524$ times the thickness of the column web, so as not to exceed the shear resistance of the column web:

$$\frac{2\phi_w 0.67 X_u}{0.66\phi\sqrt{2}} = 524$$

As an alternative to limiting the weld size, a longer stiffener may be used to reduce the shear stresses in the column web.

Example

Given:

W530x101 beam without bearing stiffener. Factored reaction = 440 kN

Web thickness = 10.9 mm, flange width = 210 mm, flange thickness = 17.4 mm

Connected to web of W310x129 column, web thickness = 13.1 mm

Design a stiffened welded seat for beams connected to both sides of the column web, which is subjected to axial compression only.

ASTM A992 grade steel is used for the beam and column, and G40.21-300W for the stiffener. Use matching electrodes, X_u = 490 MPa.

Page(s)	Revisions
3-90 to 3-91	*Replace pages 3-90 and 3-91 with the following pages.*

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 ¾ 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 × 75 × 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 ⅞ in. ASTM F3125 grade A325 bolts (or ¾ in. bolts if also used to connect the flange plates)

From Table 3-4, factored shear resistance, single shear, threads intercepted, for ⅞ in. A325 bolts = 108 kN per bolt:

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:

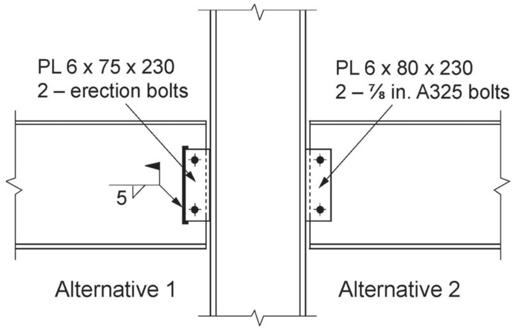
$$130 \times 10^3 / [0.75 \times 0.6 \times 230\,(300 + 440)/2] = 3.4 \text{ mm} < 6 \text{ mm}$$

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

Use a 6 × 80 × 230 plate and two ⅞ 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 × 1000 / 407 = 762 kN

Bolts

Assuming a joint length, $L < 760$ mm, from Table 3-4, the required number of ⅞ in. A325 bolts (threads excluded) is:

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

Shear per bolt = 762 / 6 = 127 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 ⅞-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).

$Z_{xe} = 1080 \times 10^3$ mm^3, $M_r = \phi Z_{xe} F_y = 335$ kN·m $> M_f = 310$ kN·m.
Other design checks may apply (e.g., Clause 13).

Factored bearing resistance, $t = 12.8$ mm

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

$$T_r = 0.75(12.8)[1.0(2 \times 35 - 26)450 + 0.6(2)(70 + 2 \times 80)(345 + 450)/2]/1000$$

$$= 1240 \text{ kN} > 762 \text{ kN}$$

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

Flange Plates

Required gross area = $762 \times 10^3 / (0.9 \times 300) = 2\,820$ mm^2 (Clause 13.2(a), tension)

Required net effective area = $762 \times 10^3 / (0.75 \times 440) = 2\,310$ mm^2

Try a 200 × 16 plate, and check areas.