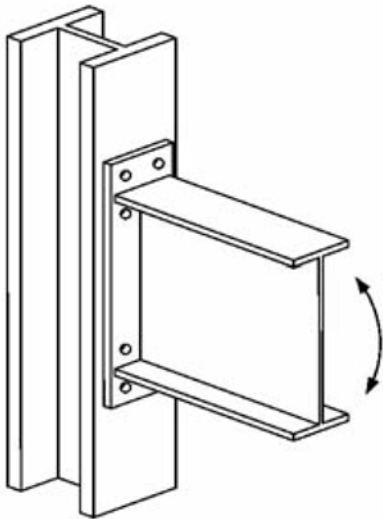
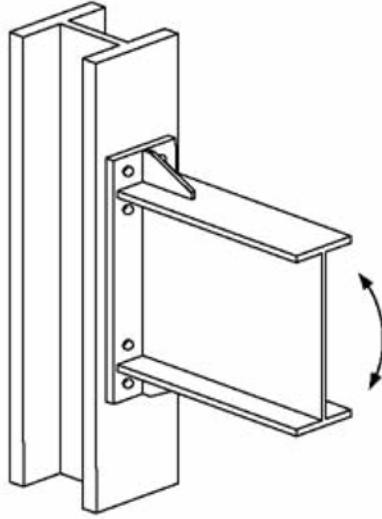


1. General

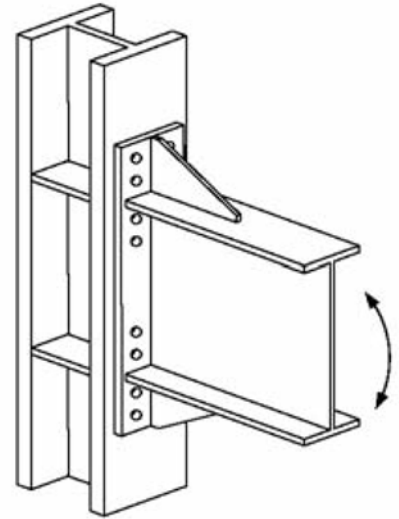
Bolted end-plate connections are made by welding the beam to an end-plate and bolting the end-plate to a column flange and design according to the chapter 6- AISC358-10. The three end-plate configurations as shown below are covered in this section and are prequalified under the AISC Seismic Provisions within the limitations of that Standard. The connection does not provide any contribution to inelastic rotation. All inelastic deformation for an end-plate connection is achieved by beam yielding and/or column panel zone deformation.



four-bolt unstiffened, 4E



four-bolt stiffened, 4ES



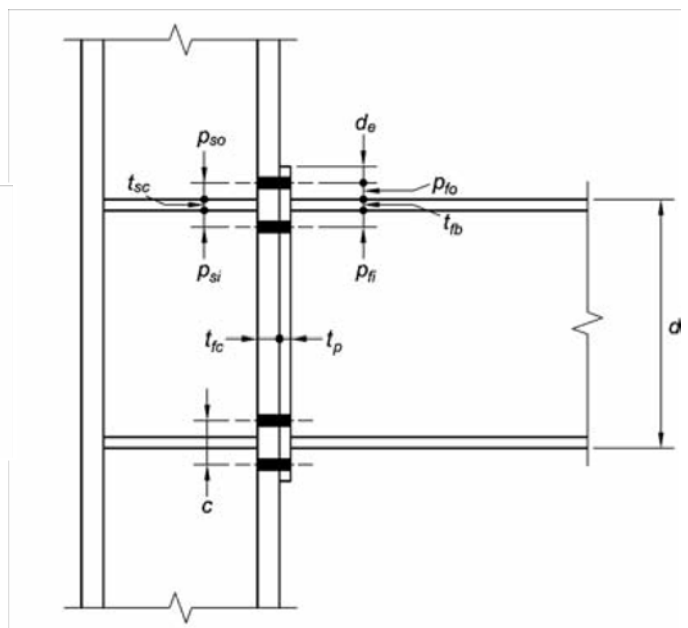
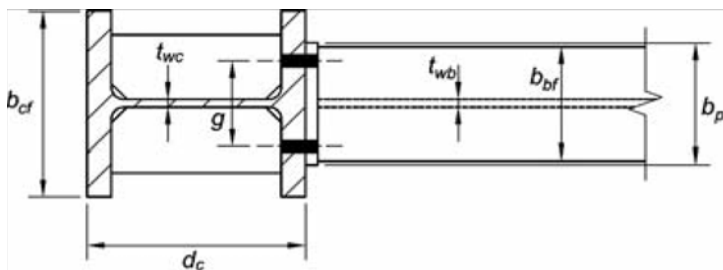
eight-bolt stiffened, 8ES.

```
select configuration type: type := "4E"
```

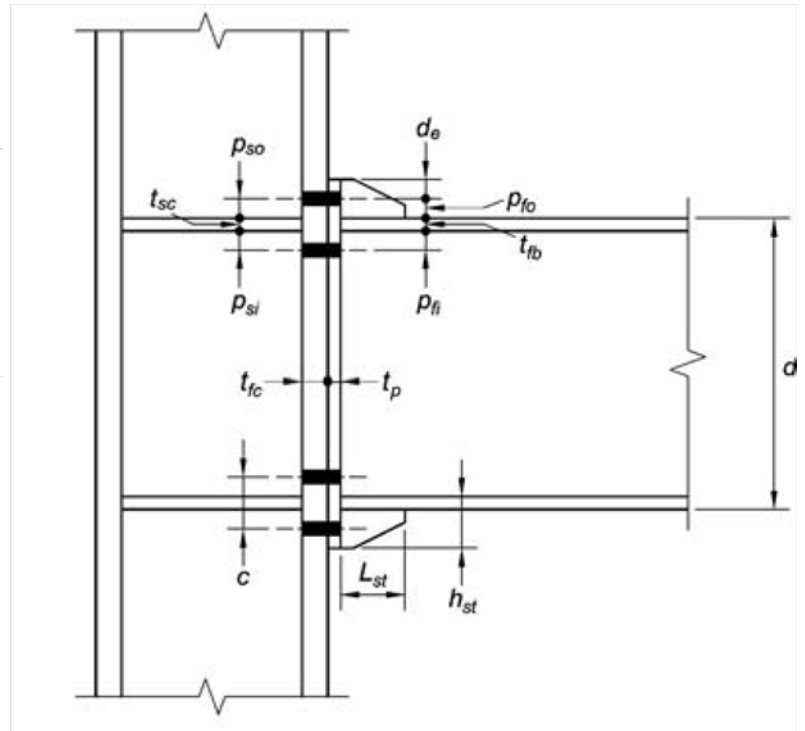
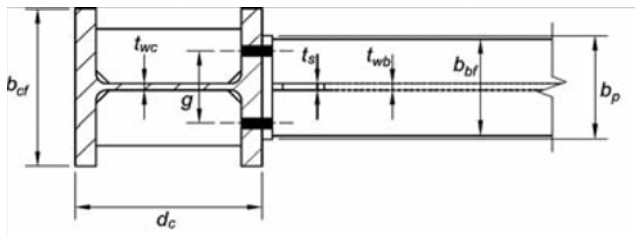
connection type: Special or Intermediate

```
connectiontype := "Intermediate"
```

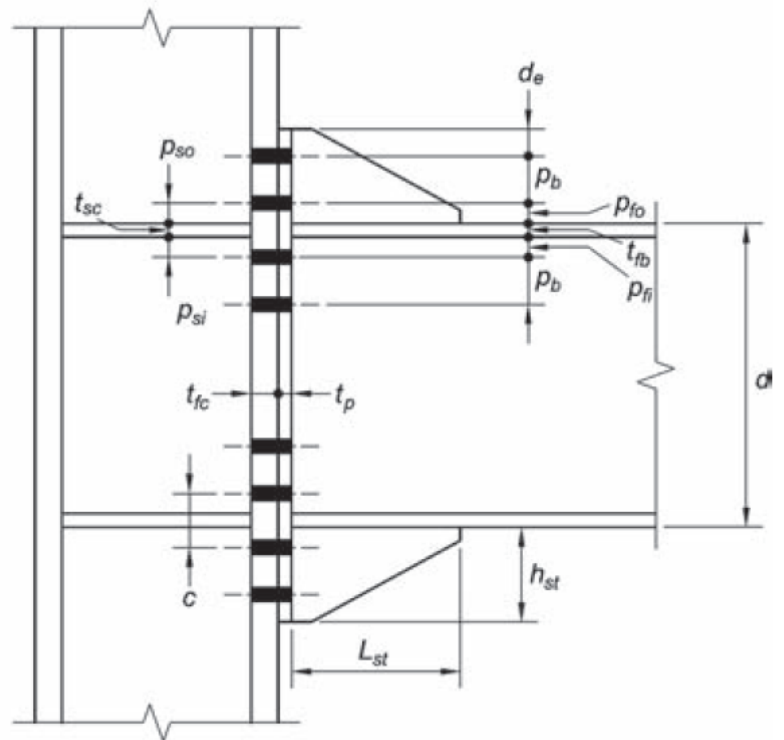
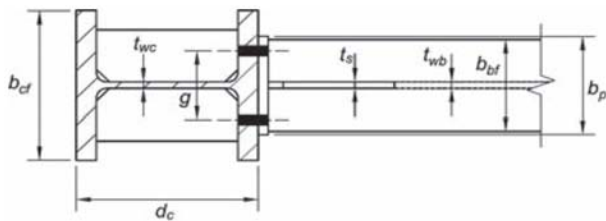
Beam and column section parameters



four-bolt unstiffened, 4E geometry



four-bolt stiffened, 4ES-geometry



eight-bolt stiffened, 8ES. geometry

beam

$b_{bf} := 30\text{cm}$	width of beam flange
$t_{bf} := 1.5\text{cm}$	thickness of beam flange
$d := 33\text{cm}$	depth of connecting beam
$t_{bw} := 0.8\text{cm}$	thickness of beam web

$$I_{bx} := \frac{1}{12} \cdot t_{bw} \cdot (d - 2 \cdot t_{bf})^3 + \frac{2}{12} \cdot t_{bf}^3 \cdot b_{bf} + 2 \cdot b_{bf} \cdot t_{bf} \cdot \left(\frac{d}{2} - \frac{t_{bf}}{2} \right)^2 = 2.414 \times 10^4 \cdot \text{cm}^4$$

$$I_{by} := 2 \cdot \frac{1}{12} \cdot t_{bf} \cdot b_{bf}^3 + \frac{1}{12} \cdot (d - 2t_{bf}) \cdot t_{bw}^3 = 6.751 \times 10^3 \cdot \text{cm}^4$$

$$S_{bx} := \frac{I_{bx} \cdot 2}{d} = 1.463 \times 10^3 \cdot \text{cm}^3$$

$$Z_{bx} := 2b_{bf} \cdot t_{bf} \cdot \left(\frac{d}{2} - \frac{t_{bf}}{2} \right) + \left(\frac{d}{2} - t_{bf} \right) \cdot t_{bw} = 1.597 \times 10^3 \cdot \text{cm}^3$$

$$A_{\text{ww}} := 2 \cdot b_{bf} \cdot t_{bf} + (d - 2t_{bf}) \cdot t_{bw} = 114 \cdot \text{cm}^2$$

bolts

$g_{\text{ww}} := 15\text{cm}$ horizontal distance between bolts

$p_{fi} := 5\text{cm}$ vertical distance from the inside of the beam tension flange to the nearest inside bolt rows

$p_{fo} := 5\text{cm}$ vertical distance from the outside of the beam tension flange to the nearest inside bolt rows

$p_{si} := 5\text{cm}$

$p_{so} := 5\text{cm}$

$d_e := 5\text{cm}$ distance from the outer bolt to the edge of the plate

$c_{\text{ww}} := p_{fo} + p_{fi} + t_{bf} = 11.5 \cdot \text{cm}$

$d_b := 20\text{mm}$ selecting bolt diameters

$d_{\text{hole}} := d_b + 2\text{mm} = 22 \cdot \text{mm}$ standard hole diameters

$A_b := \frac{\pi \cdot d_b^2}{4} = 3.142 \cdot \text{cm}^2$ Area of the one bolt

Special parameters for 8ES case

$p_b := \begin{cases} 6\text{cm} & \text{if type} = "8\text{ES}" \\ 0\text{cm} & \text{otherwise} \end{cases}$ vertical distance between the inner and outer row of bolts in an 8ES connection

end plate

$t_p := 2.5\text{cm}$ thickness of end plate

$b_p := 30\text{cm}$ width of end plate

note: The end-plate can be wider than the beam flange, but the width used in design calculations is limited to the beam flange width plus 1 in. (25 mm).

$h_p := d + 2 \cdot (p_{fo} + d_e + p_b) = 53 \cdot \text{cm}$ height of end plate

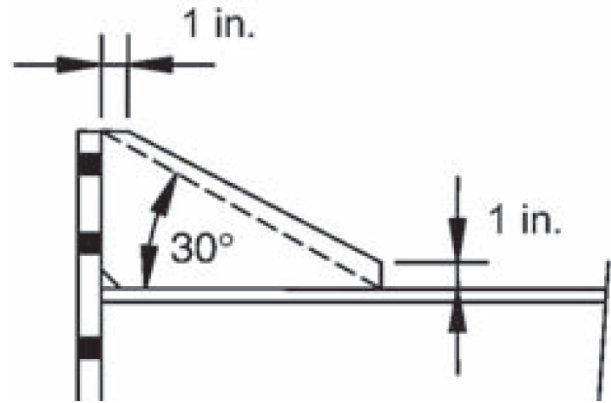
Column

$b_{cf} := 30\text{cm}$	width of column flange
$t_{cf} := 2\text{cm}$	thickness of column flange
$d_c := 33\text{cm}$	depth of column
$t_{cw} := .8\text{cm}$	thickness of column web

$$A_c := 2 \cdot b_{cf} \cdot t_{cf} + (d_c - 2t_{cf}) \cdot t_{cw} = 143.2 \cdot \text{cm}^2$$

beam stiffner

$h_{st} := d_e + p_{fo} + p_b = 10 \cdot \text{cm}$
$L_{st} := 20\text{cm}$
$t_s := 1\text{cm}$



NOTE:

According to commentary the stiffener plates shall be terminated at the beam flange and at the end of the endplate with landings approximately 1 in. (25 mm) long. The stiffener shall be clipped where it meets the beam flange and end-plate to provide clearance between the stiffener and the beam flange weld. The required 1-in. (25-mm) land is needed to ensure the quality of the vertical and horizontal weld terminations.

column stiffner

$t_{sc} := 1.5\text{cm}$	continuity plate thickness
--------------------------	----------------------------

material properties

$F_{yb} := 2400 \frac{\text{kgf}}{\text{cm}^2}$	specified minimum yield stress of the yielding element used as beam.
$F_{ub} := 3700 \frac{\text{kgf}}{\text{cm}^2}$	specified minimum tensile strength of the yielding element used as beam
$E_s := 2.1 \cdot 10^6 \frac{\text{kgf}}{\text{cm}^2}$	
$F_{yc} := 2400 \frac{\text{kgf}}{\text{cm}^2}$	specified minimum yield stress of the yielding element used as column
$F_{uc} := 3700 \frac{\text{kgf}}{\text{cm}^2}$	specified minimum tensile strength of the yielding element used as column

assumed bolt type: A490 with thread not excluded from shear plane

$F_{nt} := 780\text{MPa} = 7.954 \times 10^3 \cdot \frac{\text{kgf}}{\text{cm}^2}$	nominal tensile strength
$F_{nv} := 457\text{MPa} = 4.66 \times 10^3 \cdot \frac{\text{kgf}}{\text{cm}^2}$	nominal shear strength in bearing type connections

Threaded Parts, ksi (MPa)		
Description of Fasteners	Nominal Tensile Strength, F_{nt} , ksi (MPa) ^[a]	Nominal Shear Strength in Bearing-Type Connections, F_{nv} , ksi (MPa) ^[b]
A307 bolts	45 (310)	27 (188) ^{[c] [d]}
Group A (e.g., A325) bolts, when threads are not excluded from shear planes	90 (620)	54 (372)
Group A (e.g., A325) bolts, when threads are excluded from shear planes	90 (620)	68 (457)
Group B (e.g., A490) bolts, when threads are not excluded from shear planes	113 (780)	68 (457)
Group B (e.g., A490) bolts, when threads are excluded from shear planes	113 (780)	84 (579)
Threaded parts meeting the requirements of Section A3.4, when threads are not excluded from shear planes	$0.75F_u$	$0.450F_u$
Threaded parts meeting the requirements of Section A3.4, when threads are excluded from shear planes	$0.75F_u$	$0.563F_u$
^[a] For high-strength bolts subject to tensile fatigue loading, see Appendix 3. ^[b] For end loaded connections with a fastener pattern length greater than 38 in. (965 mm), F_{nv} shall be reduced to 83.3% of the tabulated values. Fastener pattern length is the maximum distance parallel to the line of force between the centerline of the bolts connecting two parts with one faying surface. ^[c] For A307 bolts the tabulated values shall be reduced by 1% for each $\frac{1}{16}$ in. (2 mm) over 5 diameters of length in the grip. ^[d] Threads permitted in shear planes.		

R_y = ratio of the expected yield stress to the specified minimum yield stress F_y as a in the AISC *Seismic Provisions*

TABLE A3.1 R_y and R_t Values for Steel and Steel Reinforcement Materials		
Application	R_y	R_t
Hot-rolled structural shapes and bars:		
• ASTM A36/A36M	1.5	1.2
• ASTM A1043/1043M Gr. 36 (250)	1.3	1.1
• ASTM A572/572M Gr. 50 (345) or 55 (380), ASTM A913/A913M Gr. 50 (345), 60 (415), or 65 (450), ASTM A588/A588M, ASTM A992/A992M	1.1	1.1
• ASTM A1043/A1043M Gr. 50 (345)	1.2	1.1
• ASTM A529 Gr. 50 (345)	1.2	1.2
• ASTM A529 Gr. 55 (380)	1.1	1.2
Hollow structural sections (HSS):		
• ASTM A500/A500M (Gr. B or C), ASTM A501	1.4	1.3
Pipe:		
• ASTM A53/A53M	1.6	1.2
Plates, Strips and Sheets:		
• ASTM A36/A36M	1.3	1.2
• ASTM A1043/1043M Gr. 36 (250)	1.3	1.1
• A1011/A1011M HSLAS Gr. 55 (380)	1.1	1.1
• ASTM A572/A572M Gr. 42 (290)	1.3	1.0
• ASTM A572/A572M Gr. 50 (345), Gr. 55 (380), ASTM A588/A588M	1.1	1.2
• ASTM 1043/1043M Gr. 50 (345)	1.2	1.1
Steel Reinforcement:		
• ASTM A615, ASTM A706	1.25	1.25

as material is taken from iran, R_y of St37 material is as below

$$R_y := 1$$

Span

$$L := 4.4\text{m} \quad \text{column face to column face distance}$$

$$S_h := \begin{cases} \min\left(\frac{d}{2}, 3b_{bf}\right) & \text{if type} = "4E" \\ L_{st} + t_p & \text{if type} = "4ES" \vee \text{type} = "8ES" \end{cases} \quad \text{distance from face of column to plastic hinge, in. (mm)}$$

$$L_h := L - 2S_h = 407\text{cm} \quad \text{distance between plastic hinge locations, in. (mm)}$$

2. systems

Extended end-plate moment connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems.

Exception: Extended end-plate moment connections in SMF systems with *concrete structural slabs* are prequalified only if:

-In addition to the limitations of Section 6.3-AISCE358-10, the nominal beam depth is not less than 24 in. (610 mm);

-There are no shear connectors within 1.5 times the beam depth from the face of the connected column flange

-The concrete structural slab is kept at least 1 in. (25 mm) from both sides of both column flanges. It is permitted to place compressible material in the gap between the column flanges and the concrete structural slab.

NOTE:

According to commentary design procedures are not available for connections of beams with composite action at an extended end-plate moment connection.

Therefore, careful composite slab detailing is necessary to prevent composite action that may increase tension forces in the lower bolts. Welded steel stud anchors are not permitted within 1 1/2 times the beam depth, and compressible material is required between the concrete slab and the column face (Sumner and Murray, 2002; Yang et al., 2003).

3. PREQUALIFICATION

LIMITS

Table 6.1 is a summary of the range of parameters that have been satisfactorily tested. All connection elements shall be within the ranges shown

TABLE 6.1
Parametric Limitations on Prequalification

	Four-Bolt Unstiffened (4E)		Four-Bolt Stiffened (4ES)		Eight-Bolt Stiffened (8ES)	
Parameter	Maximum in. (mm)	Minimum in. (mm)	Maximum in. (mm)	Minimum in. (mm)	Maximum in. (mm)	Minimum in. (mm)
t_{bf}	$3/4$ (19)	$3/8$ (10)	$3/4$ (19)	$3/8$ (10)	1 (25)	$9/16$ (14)
b_{bf}	$9 1/4$ (235)	6 (152)	9 (229)	6 (152)	$12 1/4$ (311)	$7 1/2$ (190)
d	55 (1400)	$13 3/4$ (349)	24 (610)	$13 3/4$ (349)	36 (914)	18 (457)
t_p	$2 1/4$ (57)	$1/2$ (13)	$1 1/2$ (38)	$1/2$ (13)	$2 1/2$ (64)	$3/4$ (19)
b_p	$10 3/4$ (273)	7 (178)	$10 3/4$ (273)	7 (178)	15 (381)	9 (229)
g	6 (152)	4 (102)	6 (152)	$3 1/4$ (83)	6 (152)	5 (127)
p_{fi}, p_{fo}	$4 1/2$ (114)	$1 1/2$ (38)	$5 1/2$ (140)	$1 3/4$ (44)	2 (51)	$1 5/8$ (41)
p_b	—	—	—	—	$3 3/4$ (95)	$3 1/2$ (89)

Only connections that are within these limits are prequalified. according to commentary of section 6-AISC358-10, there is no evidence that modest deviations from these dimensions will result in significantly different performance.

4. BEAM LIMITATIONS

-Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3. At moment-connected ends of welded built-up sections, within at least the depth of beam or 3 times the width of flange, whichever is less, the beam web and flanges shall be connected using either a complete-joint-penetration (CJP) groove weld or a pair of fillet welds each having a size 75% of the beam web thickness but not less than 1/4 in. (6 mm). For the remainder of the beam, the weld size shall not be less than that required to accomplish shear transfer from the web to the flanges.

-The clear span-to-depth ratio of the beam shall be limited as follows

(a) For SMF systems, 7 or greater.

(b) For IMF systems, 5 or greater.

$$\text{check} := \begin{cases} \text{if}\left(\frac{L}{d} \geq 5, \text{"ok"}, \text{"Not ok"}\right) & \text{if connectiontype} = \text{"Intermediate"} \\ \text{if}\left(\frac{L}{d} \geq 7, \text{"ok"}, \text{"Not ok"}\right) & \text{if connectiontype} = \text{"Special"} \end{cases}$$

check = "ok"

NOTE

according to commentary, similar to RBS testing, most of the tested beam-column assemblies had configurations approximating beam span-to-depth ratios in the range of eight to ten. However, it was judged reasonable to set the minimum span-to-depth ratio at 7 for SMF and 5 for IMF.

-Lateral bracing of beams shall be provided in accordance with the AISC *Seismic Provisions*.

AISC341-10-D-2a-(1) both flanges of beams shall be laterally braced or the beam cross section shall be torsionally braced.

AISC341-10-D-2a-(2)-beam bracing shall meet the requirements of Appendix 6 of the *Specification* for lateral or torsional bracing of beams, where the required flexural strength the required flexural strength of the member shall be:

$$M_r := R_y \cdot F_{yb} \cdot Z_{bx} = 38.34 \cdot \text{tonne} \cdot \text{m}$$

$$r_y := \sqrt{\frac{I_{by}}{A}} = 7.696 \cdot \text{cm}$$

AISC341-10-D-2a-(3)& (2b) -Beam bracing shall have a maximum spacing of

$$L_b := \begin{cases} 0.17 \cdot r_y \cdot \frac{E_s}{F_{yb}} & \text{if connectiontype} = \text{"Intermediate"} \\ 0.86 \cdot r_y \cdot \frac{E_s}{F_{yb}} & \text{if connectiontype} = \text{"Special"} \end{cases} \quad \text{maximum Beam bracing spacing}$$

$$L_b = 11.447 \cdot \text{m}$$

$$\text{check} := \text{if}(L \leq L_b, \text{"OK"}, \text{"use intermediated bracing"})$$

check = "OK"

AISC341-10-D-2c

Special bracing shall be located adjacent to expected *plastic hinge* locations where required by Chapters E, F, G or H.

(a) For structural steel beams, such bracing shall satisfy the following requirements:

(1) Both flanges of beams shall be laterally braced or the member cross section shall be torsionally braced.

(2)

The *required strength* of lateral bracing of each flange provided adjacent to plastic shall be

$$h_o := d - t_{bf} = 31.5 \cdot \text{cm} \quad \text{distance between flange centroids, in. (mm)}$$

$$P_u := \frac{0.06R_y \cdot F_{yb} \cdot Z_{bx}}{h_o} = 7.303 \cdot \text{tonne} \cdot \text{f} \quad \text{LRFD(D1-4a)}$$

The required strength of torsional bracing provided adjacent to plastic hinges shall be

$$M_u := 0.06R_y \cdot F_{yb} \cdot Z_{bx} = 2.3 \cdot \text{tonne} \cdot \text{f} \cdot \text{m} \quad \text{LRFD(D1-5a)}$$

(3) The required bracing stiffness shall satisfy the requirements of Appendix 6 of the *Specification* for lateral or torsional bracing of beams with $C_d = 1.0$ and where the expected flexural strength of the beam shall be:

$$M_r := R_y \cdot F_{yb} \cdot Z_{bx} \quad \text{LRFD(D1-6a)}$$

-The protected zone shall be determined as follows:

(a) For unstiffened extended end-plate connections: the portion of beam between the face of the column and a distance equal to the depth of the beam or 3 times the width of the beam flange from the face of the column, whichever is less.

(b) For stiffened extended end-plate connections: the portion of beam between the face of the column and a distance equal to the location of the end of the stiffener plus one-half the depth of the beam or 3 times the width of the beam flange, whichever is less.

$$\text{Protectedzone} := \begin{cases} \min(d, 3b_{bf}) & \text{if type} = "4E" \\ \min\left(L_{st} + \frac{d}{2}, 3b_{cf}\right) & \text{if type} = "4ES" \vee \text{type} = "8ES" \end{cases}$$

$$\text{Protectedzone} = 33 \cdot \text{cm}$$

5.COLUMN LIMITATIONS

-The end-plate shall be connected to the flange of the column.

-Rolled shape column depth shall be limited to W36 (W920) maximum. The depth of built-up wide-flange columns shall not exceed that for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes

-Width-to-thickness ratios for the flanges and web of the column shall conform to the requirements of the *AISC Seismic Provisions*.

NOTE:

according to commentary extended end-plate moment connections may be used only with rolled or built-up I shaped sections and must be flange connected. There are no other specific column requirements for extended end-plate moment connections

6.COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam-to-column connections shall satisfy the following limitations:

(1) Panel zones shall conform to the requirements of the *AISC Seismic Provisions*.

for intermediate moment frame

AISC341-10.(6e) Panel zone shear strength should be checked in accordance with Section J10.6 of the *Specification*. The required shear strength of the panel zone should be based

on the beam end moments computed from the load combinations stipulated by the applicable building code, not including the amplified seismic load

According to AISC-360-10.(J10.6)

The *available strength* of the web *panel zone* for the *limit state* of shear yielding shall

be determined as follows:

When required, *doubler* plate(s) or a pair of *diagonal stiffeners* shall be provided within the boundaries of the rigid connection whose webs lie in a common plane. See Section J10.9 for doubler plate design requirements.

this item is checked at last after finding required shear strength of panel zone

(2) Column-beam moment ratios shall conform to the requirements of the AISC *Seismic Provisions*

this item is checked individually

7.CONTINUITY PLATE

according to commentary continuity plate design must conform to the requirements of Section 2.4.4

Continuity plates shall satisfy the following limitations:

(1) The need for continuity plates shall be determined in accordance with Section 6.10.(AISC358-10)

(2) When provided, continuity plates shall conform to the requirements of Section 6.10.(AISC358-10)

(3) Continuity plates shall be attached to columns by welds in accordance with the AISC *Seismic Provisions*.(AISC358-10)

Exception: Continuity plates less than or equal to 3/8 in. (10 mm) shall be permitted to be welded to column flanges using double-sided fillet welds. The required strength of the fillet welds shall not be less than $F_y A_c$, where A_c is defined as the contact areas between the continuity plate and the column flanges that have attached beam flanges and F_y is defined as the specified minimum yield stress of the continuity plate.

8.Bolts

Bolts shall conform to the requirements of Chapter 4.

Note:

according to commentary on chapter 6-Prequalification tests have been conducted with both pretensioned ASTM A325 and A490 bolts. Bolt length should be such that at least two complete threads are between the unthreaded portion of the shank and the face of the nut after the bolt is pretensioned. Slip-critical connection provisions are not required for end-plate moment connections

9.CONNECTING DETAILING

-gage distance

The gage, g , is as defined in Figures 6.2 through 6.4. The maximum gage dimension is limited to the width of the connected beam flange.

check := if($g \leq b_{bf}$, "OK", "change gage distance")

check = "OK"

-Pitch and Row Spacing

The minimum pitch distance is the bolt diameter plus 1/2 in. (13 mm) for bolts up to 1 in. (25 mm) diameter, and the bolt diameter plus 3/4 in. (19 mm) for larger diameter bolts. The pitch distances, p_{fi} and p_{fo} , are the distances from the face of the beam flange to the centerline of the nearer bolt row, as shown in Figures 6.2 through 6.4. The pitch distances, p_{si} and p_{so} , are the distances from the face of the continuity plate to the centerline of the nearer bolt row, as shown in Figures 6.2 through 6.4.

-The spacing, pb , is the distance between the inner and outer row of bolts in an 8ES end-plate moment connection and is shown in Figure 6.4. The spacing of the bolt rows shall be at least 2/3 times the bolt diameter.

User Note: A distance of 3 times the bolt diameter is preferred. The distance must be sufficient to provide clearance for any welds in the region.

NOTES

-according to commentary inner bolt pitch, the distance between the face of the beam flange and the first row of inside or outside bolts, must be sufficient to allow bolt tightening. The minimum pitch values specified have been found to be satisfactory. An increase in pitch distance can significantly increase the required end-plate thickness.

-End-Plate Width

The width of the end-plate shall be greater than or equal to the connected beam flange width. The effective end-plate width shall not be taken greater than the connected beam flange plus 1 in. (25 mm).

-End-Plate Stiffener

The two extended stiffened end-plate connections, Figures 6.1(b) and (c), require a stiffener welded between the connected beam flange and the end-plate. The minimum stiffener length shall be:

$$\text{check} := \text{if} \left(L_{st} \geq \frac{h_{st}}{\tan(30^\circ)}, \text{"OK"}, \text{"Increase h.st"} \right)$$

check = "OK"

$$L_{st.min} := \frac{h_{st}}{\tan(30^\circ)} = 17.321 \cdot \text{cm}$$

When the beam and end-plate stiffeners have the same material strengths, the thickness of the stiffeners shall be greater than or equal to the beam web thickness. If the beam and end-plate stiffener have different material strengths, the thickness of the stiffener shall not be less than the ratio of the beam-to-stiffener plate material yield stresses times the beam web thickness.

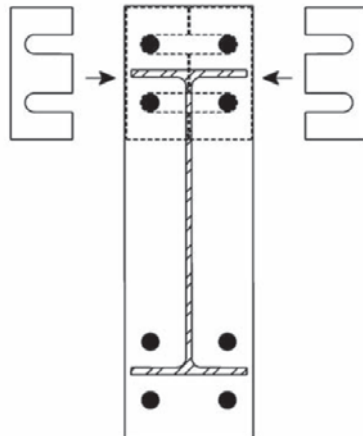
$$F_{ys} := F_{yb} = 2400 \cdot \frac{\text{kgf}}{\text{cm}^2} \quad \text{specified minimum yield stress of the yielding element used as end plate stiffener}$$

$\text{check} := \begin{cases} \text{"OK"} & \text{if } F_{yb} = F_{ys} \wedge t_s \geq t_{bw} \\ \text{"OK"} & \text{if } F_{yb} \neq F_{ys} \wedge t_s \geq t_{bw} \cdot \frac{F_{ys}}{F_{yb}} \\ \text{"increase beam stiffener thickness"} & \text{otherwise} \end{cases}$

check = "OK"

-Finger Shims

The use of finger shims (illustrated in Figure 6.6) at the top and/or bottom of the connection and on either or both sides is permitted, subject to the limitations of the RCSC *Specification*.



NOTES

-according to the commentary, tests have shown that the use of finger shims between the end-plate and the column flange do not affect the performance of the connection (Sumner et al., 2000a)

NOTES

-according to commentary, Cyclic testing has shown that use of weld access holes can cause premature fracture of the beam flange at extended end-plate moment connections (Meng and Murray, 1997). Short to long weld access holes were investigated with similar results. Therefore, weld access holes are not permitted for extended end-plate moment connections.

-according to commentary, Strain gage measurements have shown that the web plate material in the vicinity of the inside tension bolts generally reaches the yield strain (Murray and Kukreti, 1988). Consequently, it is required that the web-to-end-plate weld(s) in the vicinity of the inside bolts be sufficient to develop the strength of the beam web.

-according to commentary, The beam-flange-to-end-plate and stiffener weld requirements equal or exceed the welding that was used to prequalify the three extended end-plate moment connections. Because weld access holes are not permitted, the beam-flange-to-end plate weld at the beam web is necessarily a partial-joint-penetration (PJP) groove weld. The prequalification testing has shown that these conditions are not detrimental to the performance of the connection.

10.Design Procedure

1.End plate and bolt design

step1

probable maximum moment at plastic hinge

C_{pr} : factor to account for the peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions. Unless otherwise specifically indicated in this Standard, the value of C_{pr} shall be:

$$C_{pr} := \min\left(\frac{F_{yb} + F_{ub}}{2F_{yb}}, 1.2\right) = 1.2$$

$$C_{pr} := 1$$

ASSUMED TO 1 FOR ORDINARY MOMENT FRAME

Z_e = effective plastic section modulus of the section (or connection) at the location of the plastic hinge, in.³ (mm³)

$$Z_e := Z_{bx} = 1.597 \times 10^3 \cdot \text{cm}^3$$

$$M_{pr} := C_{pr} \cdot R_y \cdot F_{yb} \cdot Z_e = 38.34 \cdot \text{tonne} \cdot \text{m}$$

$V_{gravity}$ = beam shear force resulting from $1.2D + f_1L + 0.2S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 when the roof configuration is such that it does not shed snow off of the structure.

$$V_{gravity} := 8 \text{ tonne}$$

$$V_u := \frac{2M_{pr}}{L_h} + V_{gravity} = 26.84 \cdot \text{tonne}$$

the moment at the face of the column

$$M_{face} := M_{pr} + V_u \cdot S_h = 42.769 \cdot \text{tonne} \cdot \text{m}$$

the moment at the location of the connection

$$M_f := M_{pr} = 38.34 \cdot \text{tonne} \cdot \text{m}$$

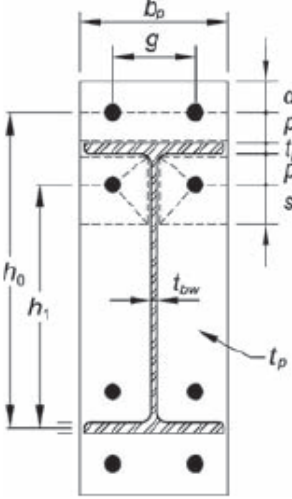
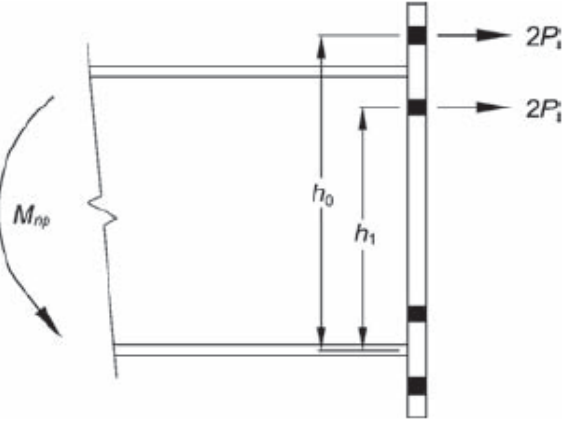
Step2

Select one of the three end-plate moment connection configurations and establish preliminary values for the connection geometry (g , pfi , pfo , pb , g , hi , etc.) and bolt grade.

Step3

Determine the required bolt diameter

TABLE 6.2
Summary of Four-Bolt Extended
Unstiffened End-Plate
Yield Line Mechanism Parameter

End-Plate Geometry and Yield Line Pattern	Bolt Force Model
	
End-Plate	$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{p_{fo}} \right) - \frac{1}{2} \right] + \frac{2}{g} [h_1 (p_{fi} + s)]$ $s = \frac{1}{2} \sqrt{b_p g} \quad \text{Note: If } p_{fi} > s, \text{ use } p_{fi} = s.$

$$h_0 := \begin{cases} \left(d - \frac{t_{bf}}{2} \right) + p_{fo} & \text{if type = "4E" } \vee \text{ type = "4ES"} \\ 0 & \text{otherwise} \end{cases}$$

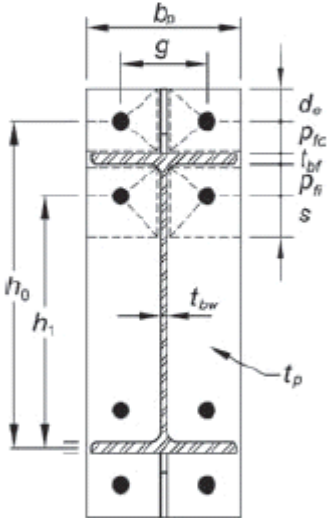
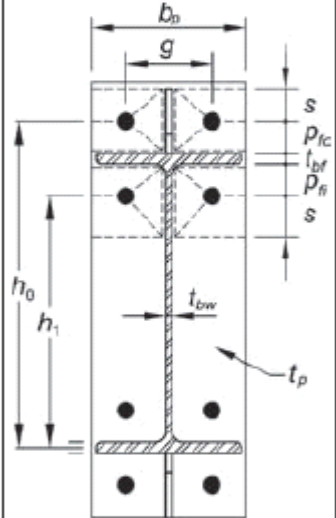
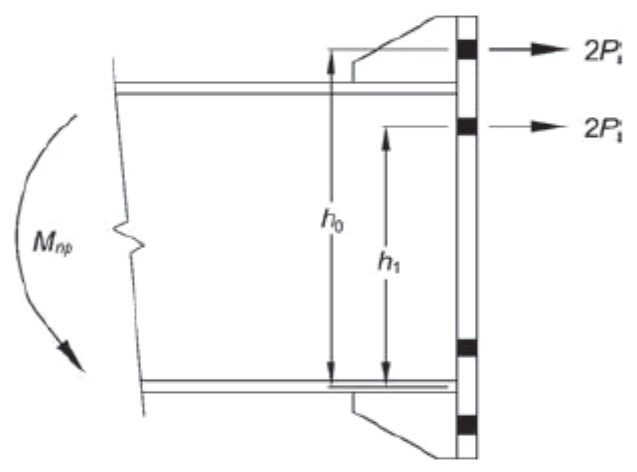
$$h_1 := \begin{cases} d - 1.5t_{bf} - p_{fi} & \text{if type = "4E" } \vee \text{ type = "4ES"} \\ 0 & \text{otherwise} \end{cases}$$

$$s := \frac{1}{2} \cdot \sqrt{b_p \cdot g} = 10.607 \cdot \text{cm}$$

$$Y_{p4E} := \begin{cases} \frac{b_p}{2} \cdot \left[h_1 \cdot \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \cdot \left(\frac{1}{p_{fo}} \right) - \frac{1}{2} \right] + \frac{2}{g} [h_1 \cdot (p_{fi} + s)] & \text{if } p_{fi} \leq s \\ \frac{b_p}{2} \cdot \left[h_1 \cdot \left(\frac{1}{s} + \frac{1}{s} \right) + h_0 \cdot \left(\frac{1}{p_{fo}} \right) - \frac{1}{2} \right] + \frac{2}{g} [h_1 \cdot (s + s)] & \text{otherwise} \end{cases}$$

$$Y_{p4E} = 2.715 \cdot \text{m}$$

TABLE 6.3
Summary of Four-Bolt Extended
Stiffened End-Plate
Yield Line Mechanism Parameter

End-Plate Geometry and Yield Line Pattern		Bolt Force Model
Case 1 ($d_e \leq s$)	Case 2 ($d_e > s$)	
		
Case 1 ($d_e \leq s$)	$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{p_{fo}} + \frac{1}{2s} \right) \right] + \frac{2}{g} [h_1(p_{fi} + s) + h_0(d_e + p_{fo})]$	
Case 2 ($d_e > s$)	$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{s} + \frac{1}{p_{fo}} \right) \right] + \frac{2}{g} [h_1(p_{fi} + s) + h_0(s + p_{fo})]$	
$s = \frac{1}{2} \sqrt{b_p g}$ Note: If $p_{fi} > s$, use $p_{fi} = s$.		

$$s := \frac{1}{2} \cdot \sqrt{b_p \cdot g} = 10.607 \cdot \text{cm}$$

$$Y_{p4ES} := \frac{b_p}{2} \cdot \left[h_1 \cdot \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \cdot \left(\frac{1}{p_{fo}} + \frac{1}{2s} \right) \right] \dots \text{ if } p_{fi} \leq s \wedge d_e \leq s$$

$$+ \frac{2}{g} [h_1 \cdot (p_{fi} + s) + h_0 \cdot (d_e + p_{fo})]$$

$$\frac{b_p}{2} \cdot \left[h_1 \cdot \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \cdot \left(\frac{1}{s} + \frac{1}{p_{fo}} \right) \right] \dots \text{ if } p_{fi} \leq s \wedge d_e > s$$

$$+ \frac{2}{g} [h_1 \cdot (p_{fi} + s) + h_0 \cdot (s + p_{fo})]$$

$$\frac{b_p}{2} \cdot \left[h_1 \cdot \left(\frac{1}{s} + \frac{1}{s} \right) + h_0 \cdot \left(\frac{1}{p_{fo}} + \frac{1}{2s} \right) \right] \dots \text{ if } p_{fi} > s \wedge d_e \leq s$$

$$+ \frac{2}{g} [h_1 \cdot (s + s) + h_0 \cdot (d_e + p_{fo})]$$

$$\frac{b_p}{2} \cdot \left[h_1 \cdot \left(\frac{1}{s} + \frac{1}{s} \right) + h_0 \cdot \left(\frac{1}{s} + \frac{1}{p_{fo}} \right) \right] \dots \text{ if } p_{fi} > s \wedge d_e > s$$

$$+ \frac{2}{g} [h_1 \cdot (s + s) + h_0 \cdot (s + p_{fo})]$$

$$Y_{p4ES} = 3.55 \text{ m}$$

TABLE 6.4
Summary of Eight-Bolt Extended
Stiffened End-Plate
Yield Line Mechanism Parameter

End-Plate Geometry and Yield Line Pattern		Bolt Force Model
Case 1 ($d_e \leq s$)	Case 2 ($d_e > s$)	

Case 1 ($d_e \leq s$)	$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{2d_e} \right) + h_2 \left(\frac{1}{p_{fo}} \right) + h_3 \left(\frac{1}{p_{fi}} \right) + h_4 \left(\frac{1}{s} \right) \right]$ $+ \frac{2}{g} \left[h_1 \left(d_e + \frac{p_b}{4} \right) + h_2 \left(p_{fo} + \frac{3p_b}{4} \right) + h_3 \left(p_{fi} + \frac{p_b}{4} \right) + h_4 \left(s + \frac{3p_b}{4} \right) + p_b^2 \right] + g$
Case 2 ($d_e > s$)	$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{s} \right) + h_2 \left(\frac{1}{p_{fo}} \right) + h_3 \left(\frac{1}{p_{fi}} \right) + h_4 \left(\frac{1}{s} \right) \right]$ $+ \frac{2}{g} \left[h_1 \left(s + \frac{p_b}{4} \right) + h_2 \left(p_{fo} + \frac{3p_b}{4} \right) + h_3 \left(p_{fi} + \frac{p_b}{4} \right) + h_4 \left(s + \frac{3p_b}{4} \right) + p_b^2 \right] + g$
$s = \frac{1}{2} \sqrt{b_p g}$ Note: If $p_{fi} > s$, use $p_{fi} = s$.	

$$s := \frac{1}{2} \cdot \sqrt{b_p \cdot g} = 10.607 \cdot \text{cm}$$

$$h_1 := \begin{cases} d - 0.5t_{bf} + p_{fo} + p_b & \text{if type} = \text{"8ES"} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot \text{cm}$$

$$h_2 := \begin{cases} d - 0.5t_{bf} + p_{fo} & \text{if type} = \text{"8ES"} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot \text{cm}$$

$$h_3 := \begin{cases} d - 1.5t_{bf} - p_{fo} & \text{if type} = \text{"8ES"} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot \text{cm}$$

$$h_4 := \begin{cases} d - 1.5t_{bf} - p_{fo} - p_b & \text{if type} = \text{"8ES"} \\ 0 & \text{otherwise} \end{cases} = 0 \cdot \text{cm}$$

$$Y_{p8ES} := \left| \begin{array}{l} \frac{b_p}{2} \cdot \left[h_1 \cdot \left(\frac{1}{2d_e} \right) + h_2 \cdot \left(\frac{1}{p_{fo}} \right) + h_3 \cdot \left(\frac{1}{p_{fi}} \right) + h_4 \cdot \left(\frac{1}{s} \right) \right] \dots \quad \text{if } p_{fi} \leq s \wedge d_e \leq s \\ + \left[\frac{2}{g} \left[h_1 \cdot \left(d_e + \frac{p_b}{4} \right) + h_2 \cdot \left(p_{fo} + 3 \frac{p_b}{4} \right) \dots \right. \right. \\ \left. \left. + \left[h_3 \cdot \left(p_{fi} + \frac{p_b}{4} \right) + h_4 \cdot \left(s + 3 \frac{p_b}{4} \right) + p_b^2 \right] \right] + g \right] \end{array} \right|$$

$$\frac{b_p}{2} \cdot \left[h_1 \cdot \left(\frac{1}{s} \right) + h_2 \cdot \left(\frac{1}{p_{fo}} \right) + h_3 \cdot \left(\frac{1}{p_{fi}} \right) + h_4 \cdot \left(\frac{1}{s} \right) \right] \dots \quad \text{if } p_{fi} \leq s \wedge d_e > s$$

$$+ \left[\frac{2}{g} \left[h_1 \cdot \left(s + \frac{p_b}{4} \right) + h_2 \cdot \left(p_{fo} + 3 \frac{p_b}{4} \right) \dots \right. \right. \\ \left. \left. + h_3 \cdot \left(p_{fi} + \frac{p_b}{4} \right) + h_4 \cdot \left(s + 3 \frac{p_b}{4} \right) + p_b^2 \right] + g \right]$$

$$\frac{b_p}{2} \cdot \left[h_1 \cdot \left(\frac{1}{2d_e} \right) + h_2 \cdot \left(\frac{1}{p_{fo}} \right) + h_3 \cdot \left(\frac{1}{p_{fi}} \right) + h_4 \cdot \left(\frac{1}{s} \right) \right] \dots \quad \text{if } p_{fi} > s \wedge d_e \leq s$$

$$+ \left[\frac{2}{g} \left[h_1 \cdot \left(d_e + \frac{p_b}{4} \right) + h_2 \cdot \left(p_{fo} + 3 \frac{p_b}{4} \right) \dots \right. \right. \\ \left. \left. + h_3 \cdot \left(p_{fi} + \frac{p_b}{4} \right) + h_4 \cdot \left(s + 3 \frac{p_b}{4} \right) + p_b^2 \right] + g \right]$$

$$\frac{b_p}{2} \cdot \left[h_1 \cdot \left(\frac{1}{s} \right) + h_2 \cdot \left(\frac{1}{p_{fo}} \right) + h_3 \cdot \left(\frac{1}{p_{fi}} \right) + h_4 \cdot \left(\frac{1}{s} \right) \right] \dots \quad \text{if } p_{fi} > s \wedge d_e > s$$

$$+ \left[\frac{2}{g} \left[h_1 \cdot \left(s + \frac{p_b}{4} \right) + h_2 \cdot \left(p_{fo} + 3 \frac{p_b}{4} \right) \dots \right. \right. \\ \left. \left. + h_3 \cdot \left(p_{fi} + \frac{p_b}{4} \right) + h_4 \cdot \left(s + 3 \frac{p_b}{4} \right) + p_b^2 \right] + g \right]$$

$$Y_{p8ES} = 0.15 \text{ m}$$

$$Y_p := \left| \begin{array}{l} Y_{p4E} \quad \text{if type} = "4E" \\ Y_{p4ES} \quad \text{if type} = "4ES" \\ Y_{p8ES} \quad \text{if type} = "8ES" \end{array} \right| \quad Y_p = 2.715 \cdot \text{m}$$

$$\phi_n := 0.9 \quad \text{resistance factor according to AISC358-10 for ductile limit state}$$

$$d_{breq} := \left| \begin{array}{l} \sqrt{\frac{2M_f}{\pi \phi_n F_{nt} (h_0 + h_1)}} \quad \text{if type} = "4E" \vee \text{type} = "4ES" \\ \sqrt{\frac{2M_f}{\pi \phi_n F_{nt} (h_1 + h_2 + h_3 + h_4)}} \quad \text{if type} = "8ES" \end{array} \right|$$

$$d_{breq} = 30.255 \cdot \text{mm}$$

$$A_{breq} := \frac{\pi \cdot d_{breq}^2}{4} \cdot 2$$

assume 4 bolt in a row

$$d_{bequivalent} := \left(\frac{A_{breq}}{4} \cdot \frac{4}{\pi} \right)^{.5} = 2.139 \cdot \text{cm}$$

h_i = distance from the centerline of the beam compression flange to the centerline of the i th tension bolt row.

h_o = distance from centerline of compression flange to the tension-side outer bolt row, in. (mm)

Step 4.

Select a trial bolt diameter, db , not less than that required in Section 6.10.1 Step 3

check := if($d_b \geq d_{bequivalent}$, "ok", "increase bolt diameters")

check = "increase bolt diameters"

Step 5.

Determine the required end-plate thickness

$\phi_d := 1$ resistance factor according to AISC358-10 for ductile limit

$F_{yp} := F_{yb} = 2400 \cdot \frac{\text{kgf}}{\text{cm}^2}$ specified minimum yield stress of the end-plate material

$F_{up} := F_{ub} = 3700 \cdot \frac{\text{kgf}}{\text{cm}^2}$

specified minimum tensile strength of the yielding element used as end-plate

Y_p = end-plate yield line mechanism parameter from Tables 6.2, 6.3 or 6.4, in. (mm)

$$t_{preq} := \sqrt{\frac{1.11 M_f}{\phi_d \cdot F_{yp} \cdot Y_p}} \quad t_{preq} = 2.556 \cdot \text{cm}$$

step 6

Select an end-plate thickness, t_p , not less than the required value

check := if($t_p \geq t_{preq}$, "OK", "increase end-plate thickness")

check = "increase end-plate thickness"

step 7

Calculate F_{fu} , the factored beam flange force

$$F_{fu} := \frac{M_f}{d - t_{bf}} = 121.714 \cdot \text{tonnef}$$

step 8

Check shear yielding of the extended portion of the four-bolt extended unstiffened end-plate (4E):

$$\text{check} := \text{if} \left(\frac{F_{fu}}{2} \leq \phi_d \cdot 0.6 F_{yp} \cdot b_p \cdot t_p, \text{"OK"} , \text{"increase tp or Fyp"} \right)$$

check = "OK"

$$\text{ratio} := \frac{\frac{F_{fu}}{2}}{\phi_d \cdot 0.6 F_{yp} \cdot b_p \cdot t_p} = 0.563$$

step 9

Check shear rupture of the extended portion of the end-plate in the four-bolt extended unstiffened end-plate (4E):

$$A_{np} := t_p \cdot [b_p - 2(d_b + 3\text{mm})] = 63.5 \cdot \text{cm}^2 \quad \text{net area of end plate}$$

$$\text{check} := \text{if} \left(\frac{F_{fu}}{2} \leq \phi_n \cdot 0.6 F_{up} \cdot A_{np}, \text{"OK"} , \text{"t.p or Fyp"} \right)$$

check = "OK"

$$\text{ratio} := \frac{\frac{F_{fu}}{2}}{\phi_n \cdot 0.6 F_{up} \cdot A_{np}} = 0.48$$

step10

select the end-plate stiffener thickness and design the stiffener-to-beam flange and stiffener-to-end-plate welds.(for 4ES and 8ES cases)

$$\text{check} := \text{if} \left(t_s \geq t_{bw} \cdot \frac{F_{yb}}{F_{ys}}, \text{"OK"} , \text{"increase stiffner thickness(ts)} \right)$$

check = "OK"

to prevent local buckling of the stiffener plate, the following width-to-thickness criterion shall be satisfied

$$\text{check} := \text{if} \left(\frac{h_{st}}{t_s} \leq 0.56 \sqrt{\frac{E_s}{F_{ys}}}, \text{"OK"} , \text{"increase thickness"} \right)$$

check = "OK"

The stiffener-to-beam-flange and stiffener-to-end-plate welds shall be designed to develop the stiffener plate in shear at the beam flange and in tension at the end-plate. Either fillet or complete-joint-penetration (CJP) groove welds are suitable for the weld of the stiffener plate to the beam flange. CJP groove welds shall be used for the stiffener-to-end-plate weld. If the end-plate is 3/8 in. (10 mm) thick or less, doublesided fillet welds are permitted.

Step11

The bolt shear rupture strength of the connection is provided by the bolts at one (compression)

flange; thus:

$$V_u = 26.84 \cdot \text{tonnef}$$

$$n_b := \begin{cases} 4 & \text{if type} = "4E" \vee \text{type} = "4ES" \\ 8 & \text{if type} = "8ES" \end{cases} \quad F_{nv} = 4.66 \times 10^3 \cdot \frac{\text{kgf}}{\text{cm}^2}$$

$$V_n := \phi_n \cdot n_b \cdot F_{nv} \cdot A_b = 52.705 \cdot \text{tonnef}$$

$$\text{check} := \text{if}(V_u \leq V_n, "OK", "change bolt type or bolt diameters")$$

$$\text{check} = "OK"$$

Step12

Check bolt-bearing/tear-out failure of the end-plate and column flange:

$$n_i := \begin{cases} 2 & \text{if type} = "4E" \vee \text{type} = "4ES" \\ 4 & \text{if type} = "8ES" \end{cases} \quad \text{number of inner bolts}$$

$$n_o := \begin{cases} 2 & \text{if type} = "4E" \vee \text{type} = "4ES" \\ 4 & \text{if type} = "8ES" \end{cases} \quad \text{number of outer bolts}$$

L_c : clear distance, in the direction of force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm)

$$\text{-for end plate:} \quad d_e - \frac{d_{\text{hole}}}{2} \quad \frac{b_p - g - d_{\text{hole}}}{2} = 0.064 \text{ m}$$

$$L_{ci} := \begin{cases} \min\left(d_e - \frac{d_{\text{hole}}}{2}, \frac{b_p - g - d_{\text{hole}}}{2}\right) & \text{if type} = "4E" \vee \text{type} = "4ES" \\ \min\left(d_e - \frac{d_{\text{hole}}}{2}, \frac{b_p - g - d_{\text{hole}}}{2}, p_b - d_{\text{hole}}\right) & \text{if type} = "8ES" \end{cases}$$

$$L_{co} := \begin{cases} \min\left(\frac{b_p - g - d_{\text{hole}}}{2}, d_e - \frac{d_{\text{hole}}}{2}\right) & \text{if type} = "4E" \vee \text{type} = "4ES" \\ \min\left(\frac{b_p - g - d_{\text{hole}}}{2}, d_e - \frac{d_{\text{hole}}}{2}, p_b - d_{\text{hole}}\right) & \text{if type} = "8ES" \end{cases}$$

$$r_{ni} := \min(1.2L_{ci} \cdot t_p \cdot F_{up}, 2.4d_b \cdot t_p \cdot F_{up}) \quad \text{for each inner bolt}$$

$$r_{no} := \min(1.2L_{co} \cdot t_p \cdot F_{up}, 2.4d_b \cdot t_p \cdot F_{up}) \quad \text{for each outer bolt}$$

$$\text{check} := \text{if}(V_u \leq \phi_n \cdot n_i \cdot r_{ni} + \phi_n \cdot n_o \cdot r_{no}, "OK", "increase clear distance")$$

$$\text{ratio} := \frac{V_u}{(\phi_n \cdot n_i \cdot r_{ni} + \phi_n \cdot n_o \cdot r_{no})} = 0.172$$

check = "OK"

-for column flange:

$$b_{cf} = 30 \cdot \text{cm}$$

$$L_{ci} := \begin{cases} \min\left(p_{fo} + p_{fi} + t_{bf}, \frac{b_{cf} - g - d_{hole}}{2}\right) & \text{if type} = "4E" \vee \text{type} = "4ES" \\ \min\left(p_{fo} + p_{fi} + t_{bf}, \frac{b_p - g - d_{hole}}{2}, p_b - d_{hole}\right) & \text{if type} = "8ES" \end{cases}$$

$$L_{co} := \begin{cases} \min\left(p_{fo} + p_{fi} + t_{bf}, \frac{b_{cf} - g - d_{hole}}{2}\right) & \text{if type} = "4E" \vee \text{type} = "4ES" \\ \min\left(p_{fo} + p_{fi} + t_{bf}, \frac{b_p - g - d_{hole}}{2}, p_b - d_{hole}\right) & \text{if type} = "8ES" \end{cases}$$

$$r_{ni} := \min(1.2L_{ci} \cdot t_{cf} \cdot F_{uc}, 2.4d_b \cdot t_{cf} \cdot F_{uc}) \quad \text{for each inner bolt}$$

$$r_{no} := \min(1.2L_{co} \cdot t_{cf} \cdot F_{uc}, 2.4d_b \cdot t_{cf} \cdot F_{uc}) \quad \text{for each outer bolt}$$

$$\text{check} := \text{if}(V_u \leq \phi_n \cdot n_i \cdot r_{ni} + \phi_n \cdot n_o \cdot r_{no}, "OK", "increase clear distance")$$

$$\text{ratio} := \frac{V_u}{(\phi_n \cdot n_i \cdot r_{ni} + \phi_n \cdot n_o \cdot r_{no})} = 0.21$$

check = "OK"

2.column-side design

Step1 Check the column flange for flexural yielding:

Y_c unstiffened column flange yield line mechanism parameter from Table 6.5
or
Table 6.6-ASCE7-10, in. (mm)

TABLE 6.5
Summary of Four-Bolt Extended
Column Flange
Yield Line Mechanism Parameter

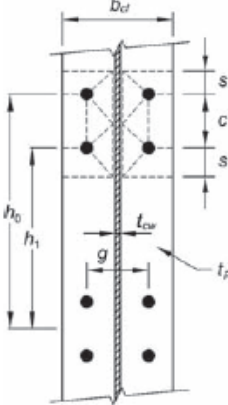
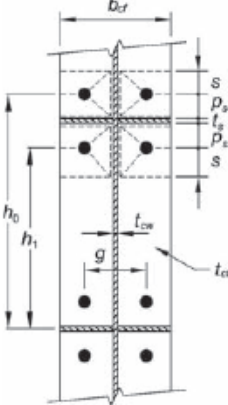
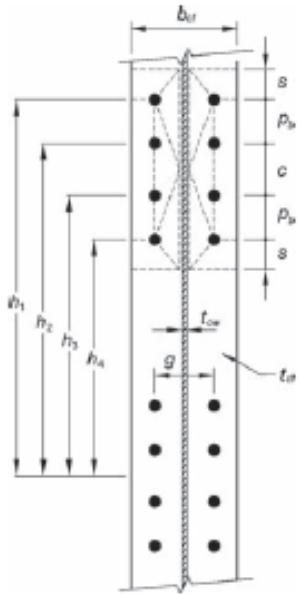
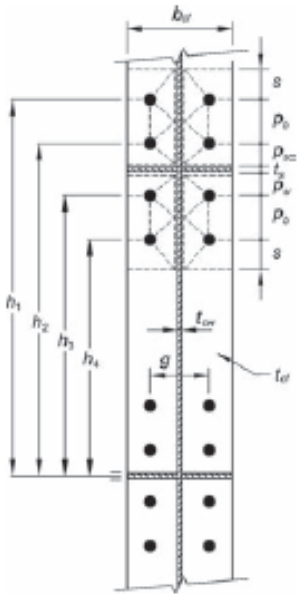
Unstiffened Column Flange Geometry and Yield Line Pattern	Stiffened Column Flange Geometry and Yield Line Pattern
	
Unstiffened Column Flange	$Y_c = \frac{b_{cf}}{2} \left[h_1 \left(\frac{1}{s} \right) + h_0 \left(\frac{1}{s} \right) \right] + \frac{2}{g} \left[h_1 \left(s + \frac{3c}{4} \right) + h_0 \left(s + \frac{c}{4} \right) + \frac{c^2}{2} \right] + \frac{g}{2}$ $s = \frac{1}{2} \sqrt{b_{cf} g}$
Stiffened Column Flange	$Y_c = \frac{b_{cf}}{2} \left[h_1 \left(\frac{1}{s} + \frac{1}{p_{si}} \right) + h_0 \left(\frac{1}{s} + \frac{1}{p_{so}} \right) \right] + \frac{2}{g} \left[h_1 (s + p_{si}) + h_0 (s + p_{so}) \right]$ $s = \frac{1}{2} \sqrt{b_{cf} g} \quad \text{Note: If } p_{si} > s, \text{ use } p_{si} = s.$

TABLE 6.6
Summary of Eight-Bolt Extended
Column Flange
Yield Line Mechanism Parameter

Unstiffened Column Flange Geometry and Yield Line Pattern	Stiffened Column Flange Geometry and Yield Line Pattern
	
Unstiffened Column Flange	$Y_c = \frac{b_{cf}}{2} \left[h_1 \left(\frac{1}{s} \right) + h_4 \left(\frac{1}{s} \right) \right] + \frac{2}{g} \left[h_1 \left(p_b + \frac{c}{s} + s \right) + h_2 \left(\frac{p_b}{2} + \frac{c}{4} \right) + h_3 \left(\frac{p_b}{2} + \frac{c}{2} \right) + h_4 (s) \right] + \frac{g}{2}$ $s = \frac{1}{2} \sqrt{b_{cf} g}$
Stiffened Column Flange	$Y_c = \frac{b_{cf}}{2} \left[h_1 \left(\frac{1}{s} \right) + h_2 \left(\frac{1}{p_{st0}} \right) + h_3 \left(\frac{1}{p_{st1}} \right) + h_4 \left(\frac{1}{s} \right) \right] + \frac{2}{g} \left[h_1 \left(s + \frac{p_b}{4} \right) + h_2 \left(p_{st0} + \frac{3p_b}{4} \right) + h_3 \left(p_{st1} + \frac{p_b}{4} \right) + h_4 \left(s + \frac{3p_b}{4} \right) + p_b^2 \right] + g$ $s = \frac{1}{2} \sqrt{b_{cf} g} \quad \text{Note: If } p_{st1} > s, \text{ use } p_{st1} = s.$

$$s := \frac{1}{2} \cdot \sqrt{b_p \cdot g} = 10.607 \cdot \text{cm}$$

$$Y_c := \begin{cases} \frac{b_{cf}}{2} \cdot \left[h_1 \cdot \left(\frac{1}{s} \right) + h_0 \cdot \left(\frac{1}{s} \right) \right] \dots & \text{if type = "4E" } \vee \text{ type = "4ES"} \\ + \left[\frac{2}{g} \cdot \left[h_1 \cdot \left(s + \frac{3c}{4} \right) + h_0 \cdot \left(s + \frac{c}{4} \right) + \frac{c^2}{2} \right] + \frac{g}{2} \right] \\ \frac{b_{cf}}{2} \cdot \left[h_1 \cdot \left(\frac{1}{s} \right) + h_4 \cdot \left(\frac{1}{s} \right) \right] \dots & \text{if type = "8ES"} \\ + \left[\frac{2}{g} \cdot \left[h_1 \cdot \left(p_b + \frac{c}{2} + s \right) + h_2 \cdot \left(\frac{p_b}{2} + \frac{c}{4} \right) + h_3 \cdot \left(\frac{p_b}{2} + \frac{c}{2} \right) + h_4 \cdot (s) \right] + \frac{g}{2} \right] \end{cases}$$

$$Y_c = 1.36 \text{ m}$$

$$\text{check} := \text{if} \left(t_{cf} \geq \sqrt{\frac{1.11 M_f}{\phi_d \cdot F_{yc} \cdot Y_c}}, "OK", "increase the column size or add continuity plate" \right)$$

check = "increase the column size or add continuity plate"

$$\text{ratio} := \frac{\sqrt{\frac{1.11 M_f}{\phi_d \cdot F_{yc} \cdot Y_c}}}{t_{cf}} = 1.806$$

Step2

If continuity plates are required for column flange flexural yielding, determine the required stiffener force.

$$M_{cf} := F_{yc} \cdot Y_c \cdot t_{cf}^2 \quad \text{The column flange flexural design strength}$$

where Y_c is the unstiffened column yield line mechanism parameter from Table 6.5

$$F_{colu1} := \frac{\phi_d \cdot M_{cf}}{d - t_{bf}}$$

the required force for continuity plate design is determined in Section 6.10.2 Step 6.

Step3-web local yeilding

Check the local column web yielding strength of the unstiffened column web at the beam flanges

$C_t = 0.5$ if the distance from the column top to the top face of the beam flange is less than the depth of the column = 1.0 otherwise

$$C_t := 1$$

k_c = distance from outer face of the column flange to web toe of fillet (design value) or fillet weld, in. (mm)

$$k_c := t_{cf} \quad \text{conservatively}$$

$$R_n := C_t \cdot (6k_c + t_{bf} + 2t_p) F_{yc} \cdot t_{cw}$$

If the strength requirement of Equation 6.10-16 is not satisfied, column web continuity plates are required.

$$\text{check} := \text{if} (F_{fu} \leq \phi_d \cdot R_n, "OK", "use column web continuity plate")$$

check = "use column web continuity plate"

$$\text{ratio} := \frac{F_{fu}}{\phi_d \cdot R_n} = 3.427$$

$$F_{colu2} := \phi_d \cdot R_n$$

$$F_{colu2} = 35.52 \cdot \text{tonnef}$$

Step4-web local buckling

Check the unstiffened column web buckling strength at the beam compression flange.

- bucklingcondition
- 1 When F_{fu} is applied at a distance greater than or equal to $dc/2$ from the end of the column
 - 2 When F_{fu} is applied at a distance less than $dc/2$ from the end of the column

bucklingcondition := 1

h is the clear distance between flanges less the fillet or corner radius for rolled shapes;

$$h := d_c - 2t_{cf}$$

$$R_n := \begin{cases} \frac{24t_{cw}^3 \cdot \sqrt{E_s \cdot F_{yc}}}{h} & \text{if bucklingcondition} = 1 \\ \frac{12t_{cw}^3 \cdot \sqrt{E_s \cdot F_{yc}}}{h} & \text{if bucklingcondition} = 2 \end{cases}$$

$$\phi := 0.75$$

$$\text{check} := \text{if}(F_{fu} \leq \phi \cdot R_n, \text{"OK"}, \text{"use column web continuity plate"})$$

$$\text{ratio} := \frac{F_{fu}}{\phi \cdot R_n} = 5.395$$

check = "use column web continuity plate"

$$F_{\text{colu3}} := \phi \cdot R_n$$

AISC-360-10-When required, a single *transverse stiffener*, a pair of transverse stiffeners, or a *doubler plate* extending the full depth of the web shall be provided

Step 5-web crippling

Check the unstiffened column web crippling strength at the beam compression flange.

- crippling condition
- 1 When F_{fu} is applied at a distance greater than or equal to $dc/2$ from the end of the column
 - 2 When F_{fu} is applied at a distance less than $dc/2$ from the end of the column

cripplingcondition := 1

N = thickness of beam flange plus 2 times the groove weld reinforcement leg size, in. (mm)

$$\text{groove}_{\text{leg}} := 0 \text{ conservatively}$$

$$N := t_{bf} + 2\text{groove}_{\text{leg}} = 1.5 \cdot \text{cm}$$

$$R_n := \begin{cases} 0.80t_{cw}^2 \cdot \left[1 + 3 \left(\frac{N}{d_c} \right) \left(\frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \cdot \sqrt{\frac{E_s \cdot F_{yc} \cdot t_{cf}}{t_{cw}}} & \text{if cripplingcondition} = 1 \\ 0.40t_{cw}^2 \cdot \left[1 + 3 \left(\frac{N}{d_c} \right) \left(\frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \cdot \sqrt{\frac{E_s \cdot F_{yc} \cdot t_{cf}}{t_{cw}}} & \text{if cripplingcondition} = 2 \wedge \frac{N}{d_c} \leq 0.2 \\ 0.40t_{cw}^2 \cdot \left[1 + \left(\frac{4N}{d_c} - 0.2 \right) \left(\frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \cdot \sqrt{\frac{E_s \cdot F_{yc} \cdot t_{cf}}{t_{cw}}} & \text{if cripplingcondition} = 2 \wedge \frac{N}{d_c} > 0.2 \end{cases}$$

$$\phi := 0.75$$

$$\text{check} := \text{if}(F_{fu} \leq \phi \cdot R_n, \text{"OK"}, \text{"use column web continuity plate"})$$

check = "use column web continuity plate"

$$\text{ratio} := \frac{F_{fu}}{\phi \cdot R_n} = 2.73$$

$$F_{\text{colu4}} := \phi \cdot R_n$$

Step 6

If stiffener plates are required for any of the column side limit states, the required strength is

The design of the continuity plates shall also conform to Chapter E of the AISC *Seismic Provisions*, and the welds shall be designed in accordance with Section 6.7(3).

AISC360-10-J10-

This section applies to *single-* and *double-concentrated forces* applied normal to the flange(s) of wide flange sections and similar *built-up shapes*. A single-concentrated force can be either tensile or compressive. Double-concentrated forces are one tensile and one compressive and form a couple on the same side of the loaded member.

When the *required strength* exceeds the *available strength* as determined for the *limit states* listed in this section, *stiffeners* and/or *doublers* shall be provided and shall be sized for the difference between the required strength and the available strength for the applicable limit state. Stiffeners shall also meet the design requirements in Section J10.8. Doublers shall also meet the design requirement in Section J10.9.

User Note: See Appendix 6.3 for requirements for the ends of cantilever members

1-flange local bending

$$\phi := 0.9$$

$$R_n := 6.25F_{yb} \cdot t_{cf}^2$$

$$F_{\text{colu5}} := \phi \cdot R_n = 54 \cdot \text{tonnef}$$

Note -

If the length of loading across the member flange is less than $0.15bf$, where bf is the member flange width, Equation J10-1 need not be checked

-When the concentrated force to be resisted is applied at a distance from the member end that is less than $10t_f$, R_n shall be reduced by 50%.

2-web local yielding

This section applies to *single-concentrated forces* and both components of *double concentrated forces*

checked according to column design step 3

3-web local crippling

checked according to column design step 5

4-web sideway buckling

This section applies only to compressive *single-concentrated forces* applied to members

where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated force

Is compression flange restrained against rotation? "Yes" or "No"

$compflangfixity := "Yes"$

M_y = yield moment corresponding to yielding of the tension flange and first yield of the compression flange, kip-in. (N-mm).

$$M_y := S_{bx} \cdot F_{yb} = 35.116 \cdot \text{tonne} \cdot \text{m}$$

M_u = required flexural strength using *LRFD load combinations*, kip-in. (N-mm)

$M_u := M_y$ assume $M_u = M_y$ conservatively

$$C_r := \begin{cases} 6.61 \cdot 10^6 \text{ MPa} & \text{if } M_u < M_y \\ 3.31 \cdot 10^6 \text{ MPa} & \text{if } M_u \geq M_y \end{cases}$$

$$C_r = 3.31 \times 10^6 \cdot \text{MPa}$$

$$h_w := d_c - 2t_{cf}$$

L_b = largest laterally *unbraced length* along either flange at the point of load, in. (mm)

$$L_b = 11.447 \text{ m}$$

$$R_n := \frac{C_r \cdot t_{cw}^3 \cdot t_{cf}}{h^2} \cdot \left[1 + 0.4 \cdot \left(\frac{\frac{h}{t_{cw}}}{\frac{L_b}{b_{cf}}} \right)^3 \right] = 55.192 \cdot \text{tonnef}$$

$$\phi := 0.85$$

$$F_{\text{colu6}} := \phi \cdot R_n = 46.913 \cdot \text{tonnef}$$

5-web compression buckling

This section applies to a pair of compressive *single-concentrated forces* or the compressive components in a pair of *double-concentrated forces*, applied at both flanges of a member at the same location.
checked according to column design step 4

6-web panel zone shear

This section applies to *double-concentrated forces* applied to one or both flanges of a member at the same location.
panel zone is checked in section 6-beam column relationship

(c) Continuity plates shall be welded to column flanges using CJP groove welds.

Continuity plates shall be welded to column webs using groove welds or fillet welds. The required strength of the sum of the welded joints of the continuity plates to the column web shall be the smallest of the following:

- (a) The sum of the *design strengths* in tension of the contact areas of the continuity plates to the column flanges that have attached beam flanges
- (b) The design strength in shear of the contact area of the plate with the column web
- (c) The design strength in shear of the column panel zone
- (d) The sum of the *expected yield strengths* of the beam flanges transmitting force to the continuity plates



AISC-341.(E2-6f)

Continuity plates shall be provided in accordance with the provisions of Section E3.6f.

AISC341-10-E3.(6.f)-(1)

(1) Continuity Plate Requirements

When the beam flange is welded to the flange of a wide-flange or built-up I-shaped column having a thickness that satisfies Equations E3-8 and E3-9, continuity plates need not be provided:

$$R_{yb} := R_y$$

$$R_{yc} := R_y$$

$$\text{check} := \text{if} \left(t_{cf} \geq 0.4 \sqrt{1.8 b_{bf} \cdot t_{bf} \cdot \frac{R_{yb} \cdot F_{yb}}{R_{yc} \cdot F_{yc}}}, "ok", "increase col flange thk or use cont pl" \right)$$

check = "increase col flange thk or use cont pl"

$$\text{check} := \text{if} \left(t_{cf} \geq \frac{b_{bf}}{6}, "ok", "increase col flange thk. or use continuity plate" \right)$$

check = "increase col flange thk. or use continuity plate"

t_{cf} = minimum required thickness of column flange when no continuity plates are provided, in. (mm)

AISC341-10-E3.(6.f)-1-3

When the beam flange is welded to the flange of the I-shape in a boxed wideflange column having a thickness that satisfies Equations E3-10 and E3-11, continuity plates need not be provided:

$$\text{check} := \text{if} \left[t_{cf} \geq 0.4 \sqrt{\left[1 - \frac{b_{bf}}{b_{cf}^2} \cdot \left(b_{cf} - \frac{b_{bf}}{4} \right) \right] \cdot \left(1.8 b_{bf} \cdot t_{bf} \cdot \frac{R_{yb} \cdot F_{yb}}{R_{yc} \cdot F_{yc}} \right)}, "ok", "increase col flange t" \right]$$

check = "ok"

$$\text{check} := \text{if} \left(t_{cf} \geq \frac{b_{bf}}{12}, "ok", "increase col flange thk. or use continuity plate" \right)$$

check = "increase col flange thk. or use continuity plate"

AISC341-10-E4.(6.f)-(2)

Continuity Plate Thickness

Where continuity plates are required, the thickness of the plates shall be determined as follows

(a) For one-sided connections, continuity plate thickness shall be at least onehalf of the thickness of the beam flange.

$$t_{\text{contplate}} := \frac{1}{2} \cdot t_{bf} = 0.75 \cdot \text{cm}$$

(b) For two-sided connections, the continuity plate thickness shall be at least equal to the thicker of the two beam flanges on either side of the column. Continuity plates shall also conform to the requirements of Section J10 of the *Specification*
this item is checked before

$$t_{cf2} := t_{cf} \quad \text{assume the section on the other side of the connection the same}$$

$$t_{\text{contplate}} := \min(t_{cf}, t_{cf2})$$

$$F_{\text{colu}} := \min(F_{\text{colu1}}, F_{\text{colu2}}, F_{\text{colu3}}, F_{\text{colu4}}, F_{\text{colu5}}, F_{\text{colu6}})$$

$$F_{\text{colu}} = 22.561 \cdot \text{tonnef}$$

minimum design strength value from Section 6.10.2 Step 2 (column flange bending), Step 3 (column web yielding), Step 4 (column web buckling), and Step 5 (column web crippling)

$$F_{\text{fu}} = 121.714 \cdot \text{tonnef} \quad \text{the factored beam flange force}$$

$$F_{\text{su}} := \begin{cases} F_{\text{fu}} - F_{\text{colu}} & \text{if } F_{\text{colu}} > 0 \\ 0 \text{kgf} & \text{if } F_{\text{colu}} = 0 \end{cases}$$

$$F_{\text{su}} = 99.153 \cdot \text{tonnef} \quad \text{continuity plate design force}$$

$$\phi_t := 0.9$$

$$A_{\text{cf}} := \frac{F_{\text{su}}}{\phi_t F_{\text{yp}}} = 45.904 \cdot \text{cm}^2$$

$$t_{\text{continuityplate}} := \frac{A_{\text{cf}}}{b_{\text{cf}}} = 1.53 \cdot \text{cm}$$

Step 7

Check the panel zone in accordance with Section 6.6(1).

see section 6.6 for beam-column relationship

AISC341-10-E3.6e Panel zone design for special moment frame

(1) required shear strength

The required shear strength of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting the expected moments at the plastic hinge points to the column faces. The *design shear strength* shall be $\phi_v R_n$ and the *allowable shear strength* shall be R_n / Ω_v where

and the *nominal shear strength*, R_n , in accordance with the limit state of shear yielding, is determined as specified in *Specification* Section J10.6.

Alternatively, the required thickness of the panel zone shall be determined in accordance with the method used in proportioning the panel zone of the tested or prequalified connection.

Is panel-zone deformation on frame stability considered in the analysis?

"Yes" or
"No"

stability_{panelzone} := "No"

P_r = required axial strength using LRFD or ASD load combinations, kips

($P_c = 0.60 P_y$, kips (N))

(ASD)

$P_y := F_{yc} \cdot A_c = 343.68 \cdot \text{tonnef}$

$P_c := 0.60 P_y = 206.208 \cdot \text{tonnef}$

P_r = required axial strength using LRFD or ASD load combinations, kips

$$P_r := 0.75P_c \quad \text{conservatively}$$

$$R_n := \begin{cases} \text{if stability}_{\text{panelzone}} = \text{"No"} \\ \left| \begin{array}{ll} 0.6F_{yc} \cdot d_c \cdot t_{cw} & \text{if } P_r \leq 0.4P_c \\ 0.6F_{yc} \cdot d_c \cdot t_{cw} \cdot \left(1.4 - \frac{P_r}{P_c} \right) & \text{if } P_r > 0.4P_c \end{array} \right. \\ \text{if stability}_{\text{panelzone}} = \text{"Yes"} \\ \left| \begin{array}{ll} 0.6F_{yc} \cdot d_c \cdot t_{cw} \cdot \left(1 + \frac{3 \cdot b_{cf} \cdot t_{cf}^2}{d \cdot d_c \cdot t_{cw}} \right) & \text{if } P_r \leq 0.75P_c \\ 0.6F_{yc} \cdot d_c \cdot t_{cw} \cdot \left(1 + \frac{3 \cdot b_{cf} \cdot t_{cf}^2}{d \cdot d_c \cdot t_{cw}} \right) \cdot \left(1.9 - \frac{1.2P_r}{P_c} \right) & \text{if } P_r > 0.75P_c \end{array} \right. \end{cases}$$

$$R_n = 24.71 \cdot \text{tonnef}$$

$$\phi := 0.9$$

$$P_{uwebpanelzone} := \phi \cdot R_n = 22.239 \cdot \text{tonnef} \quad \text{The available strength of the web panel zone for the limit state of shear yielding shall}$$

$$F_{fu} = 121.714 \cdot \text{tonnef} \quad \text{the factored beam flange force for special moment frame}$$

$$\text{check} := \text{if}(F_{fu} \leq P_{uwebpanelzone}, \text{"OK"}, \text{"panel zone thk is not enough"})$$

check = "panel zone thk is not enough"

(2)panel zone thickness

specifications for beams on two side of panel zone

beam1:

$$b_{bf1} := b_{bf} = 30 \cdot \text{cm}$$

$$t_{bf1} := t_{bf} = 1.5 \cdot \text{cm}$$

$$d_1 := d = 33 \cdot \text{cm}$$

$$t_{bw1} := t_{bw} = 0.8 \cdot \text{cm}$$

beam2:

$$b_{bf2} := b_{bf} = 30 \cdot \text{cm}$$

$$t_{bf2} := t_{bf} = 1.5 \cdot \text{cm}$$

$$d_2 := d = 33 \cdot \text{cm}$$

$$t_{bw2} := t_{bw} = 0.8 \cdot \text{cm}$$

width of beam flange

thickness of beam flange

depth of connecting beam

thickness of beam web

Column

$$b_{cf} = 30 \cdot \text{cm}$$

width of column flange

$$t_{cf} = 2 \cdot \text{cm}$$

thickness of clumn flange

$$d_c = 33 \cdot \text{cm}$$

depth of column

$$t_{cw} = 0.8 \cdot \text{cm}$$

thickness of column web

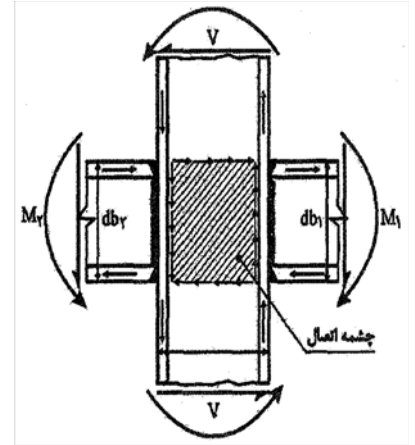
The individual thicknesses, t , of column webs and doubler plates, if used, shall conform to the following requirement:

$$d_z := \max(d_1 - 2t_{bf1}, d_2 - 2t_{bf2}) = 30 \cdot \text{cm}$$

$$w_z := d_c - 2t_{cf} = 29 \cdot \text{cm}$$

$$\text{check} := \text{if} \left(t_{cw} \geq \frac{d_z + w_z}{90 \text{mm}} \cdot \text{mm}, \text{"OK"}, \text{"use doubler plate"} \right)$$

E3.6e(2)



$$\text{ratio} := \frac{\left(\frac{d_z + w_z}{90 \text{mm}} \cdot \text{mm} \right)}{t_{cw}} = 0.819$$

check = "OK"

$$\text{condition} := \begin{cases} 1 & \text{if check} = \text{"ok"} \\ 2 & \text{otherwise} \end{cases}$$

condition = 2

$$t_{\text{dublerplate}} := 2 \text{cm}$$

added doubler plate thickness to column web in each side of column

$$\text{check} := \begin{cases} \text{if} \left(\frac{d_z + w_z}{90 \text{mm}} \cdot \text{mm} \leq t_{\text{dublerplate}}, \text{"ok"}, \text{"increase thickness of plate (t1)} \right) & \text{if condition} = 1 \\ \text{if} \left(\frac{d_z + w_z}{90 \text{mm}} \cdot \text{mm} \leq t_{cw} + 2t_{\text{dublerplate}}, \text{"ok"}, \text{"increase thickness of plate (t1)} \right) & \text{if condition} = 2 \end{cases}$$

check = "ok"

$$R_n := \begin{cases} \text{if stability}_{\text{panelzone}} = \text{"No"} \\ \left| \begin{array}{ll} 0.6F_{yc} \cdot d_c \cdot (t_{cw} + 2t_{\text{dublerplate}}) & \text{if } P_r \leq 0.4P_c \\ 0.6F_{yc} \cdot d_c \cdot (t_{cw} + 2t_{\text{dublerplate}}) \cdot \left(1.4 - \frac{P_r}{P_c}\right) & \text{if } P_r > 0.4P_c \end{array} \right. \\ \text{if stability}_{\text{panelzone}} = \text{"Yes"} \\ \left| \begin{array}{ll} 0.6F_{yc} \cdot d_c \cdot (t_{cw} + 2t_{\text{dublerplate}}) \cdot \left[1 + \frac{3 \cdot b_{cf} \cdot t_{cf}^2}{d \cdot d_c \cdot (t_{cw} + 2t_{\text{dublerplate}})}\right] & \text{if } P_r \leq 0.75P_c \\ 0.6F_{yc} \cdot d_c \cdot (t_{cw} + 2t_{\text{dublerplate}}) \cdot \left[1 + \frac{3 \cdot b_{cf} \cdot t_{cf}^2}{d \cdot d_c \cdot (t_{cw} + 2t_{\text{dublerplate}})}\right] \cdot \left(1.9 - \frac{1.2P_r}{P_c}\right) & \text{if } P_r > 0.75P_c \end{array} \right. \end{cases}$$

$$R_n = 148.262 \cdot \text{tonnef}$$

$$\phi := 0.9$$

$$P_{uwebpanelzone} := \phi \cdot R_n = 133.436 \cdot \text{tonnef}$$

$$\text{check} := \text{if}(F_{fu} \leq P_{uwebpanelzone}, \text{"OK"}, \text{"panel zone thk is not enough"})$$

check = "OK"

(3)panel zone doubler plate

Doubler plates shall be applied directly to the column web, when the web is not in compliance with Section E3.6e(2). Otherwise, doubler plates are permitted to be applied directly to the column web, or spaced away from the web.

(1)doubler plate in contact with the web

Doubler plates shall be welded to the column flanges to develop the available strength of the full doubler plate thickness, using either a complete-joint-penetration groove welded or fillet welded joint. When continuity plates are not used, the doubler plate shall be fillet welded across the top and bottom to develop the proportion of the total force that is transmitted to the doubler plate, unless the doubler plates and the web satisfy Section E3.6e(2).

(2)Spaced doubler plates

Doubler plates shall be welded to the column flanges to develop the available strength of the full doubler plate thickness, using a complete-joint-penetration groove welded joint. Doubler plates shall be placed symmetrically in pairs and located between 1/3 and 2/3 of the distance between the beam flange tip and column centerline.

(3)Doubler plates used with continuity plates

Each doubler plate shall be welded to the continuity plates to develop the proportion of the total force that is transmitted to the doubler plate.

(4) Doubler plates used without continuity plates When continuity plates are not used, doubler plates shall be extended a minimum of 6 in. (150 mm) above and below the top and bottom of the deeper moment frame beam.

User Note: When a doubler plate interferes with connecting continuity plates directly to the column web, the designer must provide a load path that satisfies the requirements of ANSI/AISC 358 Section 2.4.4b. This may be accomplished by sizing the doubler plate such that it is capable of developing the required strength of the continuity plate to column web connection. Alternatively, the doubler plate can stop inside the continuity plates. A similar load path must be provided when the web plate for a beam perpendicular to the column web connects to a doubler plate.

hk or use cont pl"]

|

= 1

dition = 2

$> 0.75P_c$