CHAPTER 6
BOLTED UNSTIFFENED AND STIFFENED EXTENDED END-PLATE MOMENT CONNECTIONS

6.1. GENERAL
Bolted end-plate connections are made by welding the beam to an end-plate and bolt- ing the end-plate to a column flange. The three end-plate configurations shown in Figure 6.1 are covered in this section and are prequalified under the AISC Seismic Provisions within the limitations of this Standard.

The behavior of this type of connection can be controlled by a number of different limit states including flexural yielding of the beam section, flexural yielding of the end-plates, yielding of the column panel zone, tension rupture of the end-plate bolts, shear rupture of the end-plate bolts, or rupture of various welded joints. The design criteria provide sufficient strength in the elements of the connections to ensure that the inelastic deformation of the connection is achieved by beam yielding.

6.2. 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:

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

![Figure 6.1. Extended end-plate configurations: (a) four-bolt unstiffened, 4E; (b) four-bolt stiffened, 4ES; (c) eight-bolt stiffened, 8ES.](image)
(2) There are no shear connectors within 1.5 times the beam depth from the face of the connected column flange; and

(3) 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.

### 6.3. 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.

### 6.4. BEAM LIMITATIONS

Beams shall satisfy the following limitations:

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Sect. 6.5. COLUMN LIMITATIONS

Columns shall satisfy the following limitations:

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

(2) 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.

(3) There is no limit on the weight per foot of columns.

(4) There are no additional requirements for flange thickness.
(5) Width-to-thickness ratios for the flanges and web of the column shall conform to the requirements of the AISC Seismic Provisions.

6.6. 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.

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

6.7. CONTINUITY PLATES

Continuity plates shall satisfy the following limitations:

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

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

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

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.

6.8. BOLTS

Bolts shall conform to the requirements of Chapter 4.

6.9. CONNECTION DETAILING

1. Gage

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.

2. 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, $p_{ji}$ and $p_{so}$, 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.
Fig. 6.2. Four-bolt unstiffened extended end-plate (4E) geometry.

Fig. 6.3. Four-bolt stiffened extended end-plate (4ES) geometry.

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The spacing, $p_b$, 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 $2\frac{2}{3}$ 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.

### 3. 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).

### 4. 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:

$$L_{st} = \frac{h_{st}}{\tan 30'} \quad (6.9-1)$$

---

Fig. 6.4. Eight-bolt stiffened extended end-plate (8ES) geometry.

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where \( h_{st} \) is the height of the stiffener, equal to the height of the end-plate from the outside face of the beam flange to the end of the end-plate as shown in Figure 6.5.

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.

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.

5. **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.

6. **Composite Slab Detailing for IMF**

In addition to the protected zone limitations, welded shear stud connectors shall not be placed along the top flange of the beam for a distance equal to 1 1/2 times the depth of the beam, measured from the face of the column.

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**Fig. 6.5. End-plate stiffener layout and geometry for 8ES. Geometry for 4ES similar.**

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**American Institute of Steel Construction**
Compressible expansion joint material, at least 1 in. (25 mm) thick, shall be installed between the slab and the column face.

7. **Welding Details**

Welding of the beam to the end-plate shall conform to the following limitations:

1. Weld access holes shall not be used.

2. The beam flange to end-plate joint shall be made using a CJP groove weld without backing. The CJP groove weld shall be made such that the root of the weld is on the beam web side of the flange. The inside face of the flange shall have a $\frac{5}{16}$-in. (8-mm) fillet weld. These welds shall be demand critical.

3. The beam web to end-plate joint shall be made using either fillet welds or CJP groove welds. When used, the fillet welds shall be sized to develop the full strength of the beam web in tension from the inside face of the flange to 6 in. (150 mm) beyond the bolt row farthest from the beam flange.

4. Backgouging of the root is not required in the flange directly above and below the beam web for a length equal to 1.5$k_1$. A full-depth PJP groove weld shall be permitted at this location.

5. When used, all end-plate-to-stiffener joints shall be made using CJP groove welds.

**Exception:** When the stiffener is $\frac{3}{8}$ in. (10 mm) thick or less, it shall be permitted to use fillet welds that develop the strength of the stiffener.

![Fig. 6.6. Typical use of finger shims.](image)
6.10. DESIGN PROCEDURE

Connection geometry is shown in Figures 6.2, 6.3 and 6.4 for the 4E, 4ES and 8ES connections, respectively.

1. End-Plate and Bolt Design

Step 1. Determine the sizes of the connected members (beams and column) and compute the moment at the face of the column, $M_f$:

$$M_f = M_{pr} + V_u S_h$$  \hspace{1cm} (6.10-1)

where

- $M_{pr}$ = probable maximum moment at plastic hinge, kip-in. (N-mm), given by Equation 2.4.3-1
- $S_h$ = distance from face of column to plastic hinge, in. (mm)
  - the lesser of $d/2$ or $3b_{bf}$ for an unstiffened connection (4E)
  - $L_{st} + t_p$ for a stiffened connection (4ES, 8ES)
- $V_u$ = shear force at end of beam, kips (N)
  $$V_u = \frac{2M_{pr}}{L_h} + V_{gravity}$$  \hspace{1cm} (6.10-2)
- $b_{bf}$ = width of beam flange, in. (mm)
- $d$ = depth of connecting beam, in. (mm)
- $L_h$ = distance between plastic hinge locations, in. (mm)
- $L_{st}$ = length of end-plate the stiffener, as shown in Figure 6.5, in. (mm)
- $t_p$ = thickness of end-plate, in. (mm)
- $V_{gravity}$ = beam shear force resulting from $1.2D + f_1L + 0.2S$ (where $f_1$ is a 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.

Step 2. Select one of the three end-plate moment connection configurations and establish preliminary values for the connection geometry ($g$, $P_{bf}$, $P_{fb}$, $P_b$, $g$, $h_i$, etc.) and bolt grade.

Step 3. Determine the required bolt diameter, $d_b \text{req}',$ using one of the following expressions.

For four-bolt connections (4E, 4ES):

$$d_b \text{req}' = \sqrt[16]{\frac{2M_f}{\pi d h_i}}$$  \hspace{1cm} (6.10-3)
For eight-bolt connections (8ES):

\[
d_b \text{ req'd} = \sqrt{\frac{2M_f}{\pi \phi_n F_{nt} (h_1 + h_2 + h_3 + h_4)}}
\]  

(6.10-4)

where

\(F_{nt}\) = nominal tensile strength of bolt from the AISC Specification, ksi (MPa)

\(h_i\) = distance from the centerline of the beam compression flange to the center-line 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, \(d_b\), not less than that required in Section 6.10.1 Step 3.

**Step 5.** Determine the required end-plate thickness, \(t_{p,\text{ req'd}}\).

\[
t_{p,\text{ req'd}} = \sqrt{\frac{1.11M_f}{\phi_d F_{yp} Y_p}}
\]  

(6.10-5)

where

\(F_{yp}\) = specified minimum yield stress of the end-plate material, ksi (MPa)

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

**Step 6.** Select an end-plate thickness, \(t_{p}\), not less than the required value.

**Step 7.** Calculate \(F_{fu}\), the factored beam flange force.

\[
F_{fu} = \frac{M_f}{d - t_{bf}}
\]  

(6.10-6)

where

\(d\) = depth of the beam, in. (mm)

\(t_{bf}\) = thickness of beam flange, in. (mm)

**Step 8.** Check shear yielding of the extended portion of the four-bolt extended unstiffened end-plate (4E):

\[
F_{fu}/2 \leq \Phi_d R_n = \Phi_d (0.6) F_{yp} b_p t_p
\]  

(6.10-7)

where \(b_p\) is the width of the end-plate, in. (mm), to be taken as not greater than the width of the beam flange plus 1 in. (25 mm).

If Equation 6.10-7 is not satisfied, increase the end-plate thickness or increase the yield stress of the end-plate material.

**Step 9.** Check shear rupture of the extended portion of the end-plate in the four-bolt extended unstiffened end-plate (4E):

\[
F_{fu}/2 \leq \Phi_h R_n = \Phi_h (0.6) F_{up} A_n
\]  

(6.10-8)
### TABLE 6.2
Summary of Four-Bolt Extended Unstiffened End-Plate Yield Line Mechanism Parameter

<table>
<thead>
<tr>
<th>End-Plate Geometry and Yield Line Pattern</th>
<th>Bolt Force Model</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="End-Plate Diagram" /></td>
<td><img src="image" alt="Bolt Force Diagram" /></td>
</tr>
</tbody>
</table>

**End-Plate**

\[
Y_p = \frac{b_p}{2} \left[ h_1 \left( \frac{1}{p_h} + \frac{1}{s} \right) + h_0 \left( \frac{1}{p_b} \right) \right] + \frac{1}{2} \left[ \frac{2}{h_1} (p_h + s) \right]
\]

\[
s = \frac{1}{2} \sqrt{b_p g}
\]

*Note: If \( p_h > s \), use \( p_h = s \).*
TABLE 6.3
Summary of Four-Bolt Extended
Stiffened End-Plate
Yield Line Mechanism Parameter

<table>
<thead>
<tr>
<th>End-Plate Geometry and Yield Line Pattern</th>
<th>Bolt Force Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1 ((d_b \leq s))</td>
<td></td>
</tr>
<tr>
<td>Case 2 ((d_b &gt; s))</td>
<td></td>
</tr>
</tbody>
</table>

\[
Y_p = \frac{b_p}{2} \left[ h_1 \left( \frac{1}{p_s} + \frac{1}{s} \right) + h_2 \left( \frac{1}{p_b} + \frac{1}{2s} \right) \right] + \frac{2}{g} \left[ h_1 (p_s + s) + h_2 (d_b + p_b) \right]
\]

\[
Y_p = \frac{b_p}{2} \left[ h_1 \left( \frac{1}{p_s} + \frac{1}{s} \right) + h_2 \left( \frac{1}{p_b} + \frac{1}{2s} \right) \right] + \frac{2}{g} \left[ h_1 (p_s + s) + h_2 (s + p_b) \right]
\]

\[
s = \frac{1}{2} \sqrt{b_p g}
\]

Note: If \(p_s > s\), use \(p_s = s\).
### TABLE 6.4
Summary of Eight-Bolt Extended Stiffened End-Plate Yield Line Mechanism Parameter

<table>
<thead>
<tr>
<th>End-Plate Geometry and Yield Line Pattern</th>
<th>Case 1 ((d_b \leq s))</th>
<th>Case 2 ((d_b &gt; s))</th>
<th>Bolt Force Model</th>
</tr>
</thead>
</table>
| <br>Case 1 \((d_b \leq s)\) | \[Y_b = \frac{b_p}{2} \left[ h_1 \left( \frac{1}{2d_b} \right) + h_2 \left( \frac{1}{p_b} \right) + h_3 \left( \frac{1}{p_b} \right) + h_4 \left( \frac{1}{s} \right) \right] \]
  + \[\frac{2}{g} \left( h_1 \left( d_b + \frac{p_b}{4} \right) + h_2 \left( p_b + \frac{3p_b}{4} \right) + h_3 \left( p_b + \frac{p_b}{4} \right) + h_4 \left( s + \frac{3p_b}{4} \right) + p_b \right) + g\] | <br>Case 2 \((d_b > s)\) | \[Y_b = \frac{b_p}{2} \left[ h_1 \left( \frac{1}{s} \right) + h_2 \left( \frac{1}{p_b} \right) + h_3 \left( \frac{1}{p_b} \right) + h_4 \left( \frac{1}{s} \right) \right] \]
  + \[\frac{2}{g} \left( h_1 \left( s + \frac{p_b}{4} \right) + h_2 \left( p_b + \frac{3p_b}{4} \right) + h_3 \left( p_b + \frac{p_b}{4} \right) + h_4 \left( s + \frac{3p_b}{4} \right) + p_b \right) + g\] |<br>

\[s = \frac{1}{2} \sqrt{b_pg}\] Note: If \(p_b > s\), use \(p_b = s\).
where

\( F_{up} \) = specified minimum tensile stress of end-plate, ksi (MPa)

\( A_n \) = net area of end-plate

\[ = t_p(b_p - 2(d_b + 1/8)) \] when standard holes are used, in.\(^2\)

\[ = t_p(b_p - 2(d_b + 3)) \] when standard holes are used, mm\(^2\)

\( d_b \) = bolt diameter, in. (mm)

If Equation 6.10-8 is not satisfied, increase the end-plate thickness or increase the yield stress of the end-plate material.

**Step 10.** If using either the four-bolt extended stiffened end-plate (4ES) or the eight-bolt extended stiffened end-plate (8ES) connection, select the end-plate stiffener thickness and design the stiffener-to-beam flange and stiffener-to-end-plate welds.

\[
t_s \geq t_{bw} \left( \frac{F_{ys}}{F_{yb}} \right) \]

(6.10-9)

where

\( t_{bw} \) = thickness of beam web, in. (mm)

\( t_s \) = end plate stiffener thickness, in. (mm)

\( F_{yb} \) = specified minimum yield stress of beam material, ksi (MPa)

\( F_{ys} \) = specified minimum yield stress of stiffener material, ksi (MPa)

The stiffener geometry shall conform to the requirements of Section 6.9.4. In addition, to prevent local buckling of the stiffener plate, the following width-to-thickness criterion shall be satisfied.

\[
\frac{h_s}{t_s} \leq 0.56 \sqrt{\frac{E}{F_{ys}}} \]

(6.10-10)

where \( h_s \) is the height of the stiffener, in. (mm), equal to the height of the end-plate from the outside face of the beam flange to the end of the end-plate.

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 \( \frac{3}{8} \) in. (10 mm) thick or less, double-sided fillet welds are permitted.

**Step 11.** The bolt shear rupture strength of the connection is provided by the bolts at one (compression) flange; thus

\[
V_u \leq \phi_n R_n = \phi_n (n_b) F_{in} A_{lb} \]

(6.10-11)
where
\( n_b \) = number of bolts at the compression flange
= 4 for 4E and 4ES connections
= 8 for 8ES connections
\( A_b \) = nominal gross area of bolt, in.\(^2\) (mm\(^2\))
\( F_{nv} \) = nominal shear strength of bolt from the AISC Specification, ksi (MPa)
\( V_u \) = shear force at the end of the beam, kips (N), given by Equation 6.10-2

**Step 12.** Check bolt-bearing/tear-out failure of the end-plate and column flange:

\[
V_u \leq \phi \sigma_n R_n = \phi \sigma_i (n_i) r_{ni} + \phi \sigma_o (n_o) r_{no}
\]  

(6.10-12)

where
\( n_i \) = number of inner bolts
= 2 for 4E and 4ES connections
= 4 for 8ES connections
\( n_o \) = number of outer bolts
= 2 for 4E and 4ES connections
= 4 for 8ES connections
\( r_{ni} = 1.2 L_c t_F u < 2.4 d_b t_F u \) for each inner bolt
\( r_{no} = 1.2 L_c t_F u < 2.4 d_b t_F u \) for each outer bolt
\( L_c \) = 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)
\( F_u \) = specified minimum tensile strength of end-plate or column flange material, ksi (MPa)
\( d_b \) = diameter of the bolt, in. (mm)
\( t \) = end-plate or column flange thickness, in. (mm)

**Step 13.** Design the flange to end-plate and web to end-plate welds using the requirements of Section 6.9.7.

2. **Column-Side Design**

**Step 1.** Check the column flange for flexural yielding:

\[
t_{cf} \geq \frac{1.11 M_f}{\phi_i F_{yc} Y_c}
\]  

(6.10-13)

where
\( F_{yc} \) = specified minimum yield stress of column flange material, ksi (MPa)
\( Y_c \) = unstiffened column flange yield line mechanism parameter from Table 6.5 or Table 6.6, in. (mm)
\( t_{cf} \) = column flange thickness, in. (mm)
### TABLE 6.5
Summary of Four-Bolt Extended Column Flange Yield Line Mechanism Parameter

<table>
<thead>
<tr>
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<th>Stiffened Column Flange Geometry and Yield Line Pattern</th>
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</thead>
<tbody>
<tr>
<td><img src="image1" alt="Unstiffened Column Flange Diagram" /></td>
<td><img src="image2" alt="Stiffened Column Flange Diagram" /></td>
</tr>
<tr>
<td><strong>Unstiffened Column Flange</strong></td>
<td></td>
</tr>
<tr>
<td>$Y_c = \frac{b_w}{2} \left[ h_1 \left( \frac{1}{s^2} \right) + h_b \left( \frac{1}{s} \right) \right] + \frac{2}{g} \left[ h_1 \left( s + \frac{3c}{4} \right) + h_b \left( s + \frac{c}{4} \right) + \frac{c^2}{2} \right] + \frac{g}{2}$</td>
<td></td>
</tr>
<tr>
<td>$s = \frac{1}{2} \sqrt{b_w g}$</td>
<td></td>
</tr>
<tr>
<td><strong>Stiffened Column Flange</strong></td>
<td></td>
</tr>
<tr>
<td>$Y_c = \frac{b_w}{2} \left[ h_1 \left( \frac{1}{s} + \frac{1}{r_w} \right) + h_b \left( \frac{1}{s} + \frac{1}{r_w} \right) \right] + \frac{2}{g} \left[ h_1 \left( s + p_w \right) + h_b \left( s + p_w \right) \right]$</td>
<td></td>
</tr>
<tr>
<td>$s = \frac{1}{2} \sqrt{b_w g}$</td>
<td>Note: If $p_w &gt; s$, use $p_w = s.$</td>
</tr>
</tbody>
</table>

Note: If $\psi > s$, use $\psi = s.$
### TABLE 6.6
Summary of Eight-Bolt Extended Column Flange Yield Line Mechanism Parameter

<table>
<thead>
<tr>
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</table>

#### Unstiffened Column Flange

\[
Y_c = \frac{b_c f_0}{2} \left[ h_1 \left( \frac{1}{s} \right) + h_4 \left( \frac{1}{s} \right) \right] + \frac{2}{g} \left[ h_2 \left( \frac{p_b + c + s}{g} \right) + h_3 \left( \frac{p_b + c}{2} \right) + h_4 (s) \right] \frac{g}{2}
\]

\[
s = \frac{1}{2} \sqrt{b_c \sigma_f}
\]

#### Stiffened Column Flange

\[
Y_c = \frac{bh}{2} \left[ h_1 \left( \frac{1}{s} \right) + h_2 \left( \frac{1}{p_{b}} + h_3 \left( \frac{1}{p_{b}} + h_4 \left( \frac{1}{s} \right) \right) \right] + \frac{2}{g} \left[ h_4 \left( s + \frac{p_b}{4} \right) + h_3 \left( \frac{p_b}{4} \right) + h_4 \left( s + \frac{p_b}{4} \right) \right] + g
\]

\[
s = \frac{1}{2} \sqrt{b_c \sigma_f}
\]

Note: If \( p_{b} \) > \( s \), use \( p_{b} = s \).
If Equation 6.10-13 is not satisfied, increase the column size or add continuity plates.

If continuity plates are added, check Equation 6.10-13 using $Y_c$ for the stiffened column flange from Tables 6.5 and 6.6.

**Step 2.** If continuity plates are required for column flange flexural yielding, determine the required stiffener force.

The column flange flexural design strength is

$$
\phi_d M_{cf} = \phi_d F_{yc} Y_c t_{cf}^2
$$

(6.10-14)

where $Y_c$ is the unstiffened column yield line mechanism parameter from Table 6.5 or Table 6.6, in. (mm). Therefore, the equivalent column flange design force is

$$
\phi_d R_n = \frac{\phi_d M_{cf}}{d - t_{bf}}
$$

(6.10-15)

Using $\phi_d R_n$, the required force for continuity plate design is determined in Section 6.10.2 Step 6.

**Step 3.** Check the local column web yielding strength of the unstiffened column web at the beam flanges.

Strength requirement:

$$
F_{fu} \leq \phi_d R_n
$$

(6.10-16)

$$
R_n = C_t (6k_c + t_{bf} + 2t_f) F_{yc} t_{cw}
$$

(6.10-17)

where

$C_t = 0.5$ 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

$F_{yc}$ = specified yield stress of column web material, ksi (MPa)

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

$t_{cw}$ = column web thickness, in. (mm)

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

**Step 4.** Check the unstiffened column web buckling strength at the beam compression flange.

Strength requirement:

$$
F_{ju} \leq \phi R_u
$$

(6.10-18)

where $\phi = 0.75$
(a) When $F_{fu}$ is applied at a distance greater than or equal to $d_c/2$ from the end of the column

$$R_n = \frac{24t_{cw}^3 \sqrt{EF_{yc}}}{h}$$  \hspace{1em} (6.10-19)

(b) When $F_{fu}$ is applied at a distance less than $d_c/2$ from the end of the column

$$R_n = \frac{12t_{cw}^3 \sqrt{EF_{yc}}}{h}$$  \hspace{1em} (6.10-20)

where $h$ is the clear distance between flanges less the fillet or corner radius for rolled shapes; clear distance between flanges when welds are used for built-up shapes, in. (mm)

If the strength requirement of Equation 6.10-18 is not satisfied, then column web continuity plates are required.

**Step 5.** Check the unstiffened column web crippling strength at the beam compression flange.

Strength requirement:

$$F_{fu} \leq \phi R_n$$  \hspace{1em} (6.10-21)

where $\phi = 0.75$

(a) When $F_{fu}$ is applied at a distance greater than or equal to $d_c/2$ from the end of the column

$$R_n = 0.80 t_{cw}^2 \left[ 1 + 3 \left( \frac{N}{d_c} \right) \left( \frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \sqrt{\frac{EF_{yc}t_{cf}}{t_{cw}}}$$  \hspace{1em} (6.10-22)

(b) When $F_{fu}$ is applied at a distance less than $d_c/2$ from the end of the column

(i) for $N/d_c \leq 0.2$,

$$R_n = 0.40 t_{cw}^2 \left[ 1 + 3 \left( \frac{N}{d_c} \right) \left( \frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \sqrt{\frac{EF_{yc}t_{cf}}{t_{cw}}}$$  \hspace{1em} (6.10-23)

(ii) for $N/d_c > 0.2$,

$$R_n = 0.40 t_{cw}^2 \left[ 1 + \left( \frac{4N}{d_c} - 0.2 \right) \left( \frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \sqrt{\frac{EF_{yc}t_{cf}}{t_{cw}}}$$  \hspace{1em} (6.10-24)
where

\[ N = \text{thickness of beam flange plus 2 times the groove weld reinforcement leg size, in. (mm)} \]
\[ d_c = \text{overall depth of the column, in. (mm)} \]

If the strength requirement of Equation 6.10-21 is not satisfied, then column web continuity plates are required.

**Step 6.** If stiffener plates are required for any of the column side limit states, the required strength is

\[ F_{su} = F_{fu} - \min(\phi R_n) \quad (6.10-25) \]

where \( \min(\phi R_n) \) is the 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).

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).

**Step 7.** Check the panel zone in accordance with Section 6.6(1).