Moment End Plate – Column Side

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Introduction
Standard moment end plate connections (MEP) have been developed by Steel Construction New Zealand Inc. The design procedures are presented in Structural Steelwork Connections Guide: Design Procedures, SCNZ 14-1:2007 (Hyland et al, 2008). The connection details are given in table form in SCNZ 14-2:2007 (Hyland et al, 2008). However these publications only provide details for the beam side of the MEP connection. The column side aspects are not covered and reference is given to guidance in HERA Report R4-142:2007 (Clifton et al, 2007). This article updates the guidance and design example given in this publication and presents the design procedure consistent with the SCNZ publications. Development of software based on these procedures is being developed and this is briefly discussed.

![Figure 1: Moment End Plate Connection](image)

Design Procedure
Step 1: Design Continuity Stiffeners
This design procedure is based on the presence of continuity stiffeners in the column, aligned with the incoming beam flanges. The presence of the continuity stiffeners increases the tension pull out capacity of the column flange.

The Steel Structures Standard NZS 3404 cl 12.9.5.3.1(c) (SNZ, 2007) presents two equations for determining the area of continuity stiffeners. There is an equation for the continuity stiffener opposite the compression flange and an equation for when the continuity stiffener is opposite the tension flange. Under seismic actions the loading is reversing. The governing equation is the equation for the continuity stiffener opposite the tension flange. Using this equation the total area of the continuity stiffeners is to be at least the equivalent area of the beam flange adjusted for the difference in yield stress. This is reproduced as equation 1. This equation is based on the overstrength moment action being developed in the beam.

\[
A_{\text{spair}} \geq \eta t_f - t_{wc} t_f \left( \frac{f_{yd}}{t_f} \right)
\]

(1)

where:
\[b_f = \text{width of beam flange}\]

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$t_f$ = thickness of beam flange
$t_{wc}$ = thickness of column web
$f_{yf}$ = yield stress of beam flange
$f_{ys}$ = yield stress of continuity stiffener

Suppressing local buckling of the continuity stiffener in compression is achieved by limiting the ratio of the stiffener outstand width to stiffener thickness. Note 1 from clause 12.9.5.3 of NZS 3404 (SNZ, 2007) states two different ratios of stiffener outstand width / thickness limits. There is one limit for connections in which the incoming beams are category 1 or 2 members and a less restrictive limit for connections in which the incoming beams are category 3 or 4. This is reproduced as equation 2. For category 1 or 2 incoming beam and stiffener yield stress of 250 MPa gives a maximum outstand width of 8 times stiffener thickness.

$$t_s \geq \left( \frac{b_s}{C_1} \right) \left( \frac{f_{ys}}{250} \right)$$ (2)

where:
$C_1$ = 8 for incoming beam being a category 1 or 2 member
$C_1$ = 15 otherwise

The width of the continuity stiffer is selected such that the combine stiffener widths and column web is greater than the beam flange but not greater than the column flange width. This is given in equation 3. Standard flat widths should be selected where possible to minimise fabrication costs.

$$\frac{b_f - t_{wc}}{2} \leq b_s \leq \frac{b_{fc} - t_{wc}}{2}$$ (3)

where:
$b_{fc}$ = width of column flange

For rotary-straightened hot rolled I sections an area of reduced notch toughness has been found in a limited region of web adjacent to the flange. (AISC, 2005) This is of particular issue for heavy sections. Therefore for heavy I sections of columns in seismic moment resisting systems welding in the k-area is to be avoided.

Corners of continuity stiffener plates placed in the webs of rolled sections with elements greater than 32 mm thick should be clipped to avoid the k-areas (Refer Figure 2) as follows. Along the web, the clip should extend a distance of 35 mm beyond the tangent of the web-to-flange radius. Along the flange, the clip should extend 12 mm beyond the tangent of the web-to-flange radius. The welds to the continuity stiffener should be terminated 5 mm back from the clipped corners.

Welds of continuity stiffeners to column webs and column flange are to be designed to develop the design capacity of the continuity stiffener. The weld of the continuity stiffener remote from an incoming beam may be welded with a nominal fillet weld.

![Figure 2: Continuity Stiffener Plates for Heavy Sections](image)
**Step 2: Determine if Column Flange Backing Plates are required**
Typically if the column flange thickness is less than the beam end plate thickness then backing plates are required. Backing plates are only effective for mode 1 failure as described below.

**Step 3: Determine column flange tension equivalent tee stub length**
The complex pattern of yield lines which occurs round the bolts is converted into a simple ‘equivalent tee-stub’. This is illustrated in figure 3.

![Figure 3: Equivalent tee stub concept (SCI, 1995)](image)

For the first bolt row, above the continuity stiffener, there are 3 principle patterns of yield lines. This is circular yielding, side yielding and side yielding near a stiffener as illustrated in figure 4. Two other cases are when the connection is at the top of the column. These are corner yielding and corner yielding near a stiffener.

![Figure 4: Patterns of yielding (SCI, 1995)](image)

For bolt row 2 below the stiffener the yield lines patterns are the same 3 principle patterns above. However the resultant effective length must take into account the presence of the 3rd bolt row and reduce the effective tee length as required.

For the 3rd bolt row the 2 principle yielding patterns are circular yielding and side yielding. But the resultant effective tee stub length must take into account the presence of the 2nd bolt row.
The backing plates are detailed such that the same effective tee stub lengths can be taken for each bolt row. This is the method taken in Eurocode 3 (BS EN 1993-1-8, 2005). The presence of the backing plate increases the effective tee stub length for the column flange (Clifton et al, 2007).

Table 1: Effective Tee Stub Lengths

<table>
<thead>
<tr>
<th>Bolt row location</th>
<th>Yielding pattern effective tee stub length</th>
<th>Resultant effective tee stub length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top bolt row above continuity stiffener</td>
<td>Circular pattern ( l_{e1c} = 2\pi m_4 )</td>
<td>Column flange ( l_{e2c} = \left( l_{e2c} - l_{e3c} \right) \max + t_{bp} )</td>
</tr>
<tr>
<td></td>
<td>Side yielding ( l_{e2c} = 4m_4 + 1.25e_c )</td>
<td>Backing plate ( l_{e1bp} = \left( l_{e1bp} - l_{e2bp} \right) \max )</td>
</tr>
<tr>
<td></td>
<td>Side yielding near stiffener ( l_{e3c} = \alpha_c m_4 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Corner yielding ( l_{e5c} = 2m_4 + 0.625e_c + a_{e5c} )</td>
<td>Second of two rows column flange ( l_{e2c} = \left( l_{e2c} - l_{e3c} \right) \max + t_{bp} )</td>
</tr>
<tr>
<td></td>
<td>Corner yielding near stiffener ( l_{e6c} = \alpha_c m_4 - (2m_4 + 0.625e_c) + a_{e6c} )</td>
<td>Second of two rows column flange backing plate ( l_{e2bp} = \left( l_{e2bp} - l_{e3bp} \right) \max )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Second of three rows column flange ( l_{e2c} = \left( l_{e2c} - l_{e3c} \right) \max + 0.5s_{p \ min} + t_{bp} )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Second of three rows column flange backing plate ( l_{e2bp} = \left( l_{e2bp} - l_{e3bp} \right) \max + 0.5s_{p \ min} )</td>
</tr>
<tr>
<td>Second bolt row, immediately below continuity stiffener</td>
<td>Circular yielding pattern ( l_{e1c} = 2\pi m_3c )</td>
<td>Column flange ( l_{e3c} = \left( l_{e3c} - l_{e2c} \right) \max + 0.5s_{p \ min} )</td>
</tr>
<tr>
<td></td>
<td>Side yielding pattern ( l_{e2c} = 4m_3c + 1.25e_c )</td>
<td>Backing Plate ( l_{e3bp} = \left( l_{e3bp} - l_{e2bp} \right) \max )</td>
</tr>
<tr>
<td></td>
<td>Side yielding near stiffener pattern ( l_{e3c} = \alpha_c m_3c )</td>
<td></td>
</tr>
<tr>
<td>Third bolt row</td>
<td>Circular yielding pattern ( l_{e1c} = 2\pi m_2c )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Side yielding pattern ( l_{e2c} = 4m_2c + 1.25e_c )</td>
<td></td>
</tr>
<tr>
<td></td>
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</tr>
</tbody>
</table>

Note: Definition of terms may be found in the appendix

**Step 4: Determine column flange tension capacity for each bolt row**
For each bolt row the design tension capacity is given by the minimum of modes 1, 2 and 3. The 3 modes are illustrated in figure 5. Mode 1 is complete flange yielding; no bolt extension. Mode 2 is partial flange yielding; partial bolt extension. Mode 3 is no flange yielding; full bolt extension.

Figure 5: 3 Modes of Failure (Hyland et al, 2008)
The equations for determining tension capacity for each mode are as follows:

**Mode 1:**
\[
\phi N_{tc} = \frac{4\phi M_{pc} + 2\phi M_{bp}}{m_c} \tag{4}
\]

This can be written as:
\[
\phi N_{tc} = \frac{\phi_s f_{yfc} l_{erxc} t^2_c + 0.5\phi_s f_{yb} l_{erbpx} t^2_{bp}}{m_c} \tag{5}
\]

where:
- \(\phi M_{pc}\) = design plastic moment capacity of equivalent column flange tee stub
- \(\phi M_{bp}\) = design plastic moment capacity of equivalent backing plate tee stub
- \(m_c\) = bolt distance from column web root, defined in appendix
- \(l_{erxc}\) = effective column flange tee stub length
- \(t_{fc}\) = column flange thickness
- \(f_{yfc}\) = column flange yield stress
- \(l_{erbpx}\) = effective backing plate tee stub length
- \(t_{bp}\) = backing plate thickness
- \(f_{yb}\) = backing plate yield stress

**Mode 2:**
\[
\phi N_{tc} = \frac{2\phi M_{pc} + n_c 2\phi_s N_{tf}}{m_c + n_c} \tag{6}
\]

This can be written as:
\[
\phi N_{tc} = \frac{0.5\phi_s f_{yfc} l_{erxc} t^2_c + n_c 2\phi_s N_{tf}}{m_c + n_c} \tag{7}
\]

where:
- \(n_c\) = effective edge distance, defined in appendix
- \(\phi_s N_{tf}\) = bolt tension design capacity

**Mode 3:**
\[
\phi N_3 = 2\phi_s N_{tf} \tag{8}
\]

**Step 5: Determine column web tension capacity**

The continuity stiffeners have been designed to have the equivalent area of the beam flange. Because of this there is no need to check column web tension capacity for bolt row 1 and 2. For bolt row 3 the effective length of the column web in tension is calculated as illustrated in figure 6.

\[
l_{13} = \left(0.9 s_g, \frac{S_p}{2}\right)_{\min} + 0.9 s_g \tag{10}
\]

where:
- \(s_g\) = bolt gauge

The column web tension capacity for bolt row 3 is then:
\[
\phi N_{3w} = \phi_t l_{13} t_{wc} f_{yw} \tag{11}
\]

Over the effective tension length the column web to column flange connection must be able to develop the design tension capacity of the column web. For a hot-rolled column, this is always satisfied. For a three plate welded column, the weld size must be checked.
Step 6: Calculate moment capacity
The moment capacity is the minimum bolt row tension capacity times the lever arm.

\[
\phi M_r = \phi N_{r1} \phi N_{r1c} \frac{d_1}{L_{min}} + \phi N_{r2} \phi N_{r2c} \frac{d_2}{L_{min}} + \psi_{r3} \phi N_{r3} \phi N_{r3c} \frac{d_3}{L_{min}}
\]  

(12)

where:
- \( \phi N_{r1} \) = Bolt row 1 design tension capacity on beam side, from (Hyland et al, 2008)
- \( \phi N_{r2} \) = Bolt row 2 design tension capacity on beam side, from (Hyland et al, 2008)
- \( \phi N_{r3} \) = Bolt row 3 design tension capacity on beam side, from (Hyland et al, 2008)
- \( d_1 \) = lever arm of bolt row 1, from (Hyland et al, 2008)
- \( d_2 \) = lever arm of bolt row 2, from (Hyland et al, 2008)
- \( d_3 \) = lever arm of bolt row 3, from (Hyland et al, 2008)
- \( \psi_{r3} \) = bolt row capacity reduction for plastic distribution limits, from (Hyland et al, 2008)

Step 7: Detail backing plates
The width of the backing plate should not be less than the distance from the edge of the flange to the toe of the root radius and should fit snugly against the root radius.

The length of the backing plate is detailed such that the effective tee stub lengths determined above can be achieved. The following bolt edge distances must be met.

\[
a_{ebp1} \geq [2d_f, 0.25, a_{etl}]_{max}
\]

\[
a_{ebp2} \geq [2d_f, 0.25, a_{etl}]_{max}
\]

\[
a_{ebp3} \geq [2d_f, 0.5, a_{etl}]_{max}
\]

where:
- \( d_f \) = bolt diameter
- \( a_{etl} \) = edge distance to a free end

Step 8: Determine column flange bolt bearing capacity
The column flange bolt bearing capacity must be checked if the column flange thickness is less than the beam end plate thickness. The column flange bolt bearing capacity is given as

\[
\phi V_{fc} = n_{ub} 3.2 f_{ucf} d_f t_{fc}
\]

(13)

where:
- \( n_{ub} \) = number of bottom bolts
- \( f_{ucf} \) = Column flange tensile strength
**Step 9: Determine column panel zone shear action**

Column panel zone shear action is determined in accordance with NZS 3404. NZS 3404 cl 12.9.5.2 (SNZ, 2007) sets out procedure for determining design actions from beams for the joint panel zone. The equation is:

\[
V_p^* = \frac{M_L}{(d_b - t_f)_L} + \frac{M_R}{(d_b - t_f)_R} - V_{Col} - V_c
\]

Where

1. The subscripts L and R denote the left and right hand beams at the connection.
2. \(V_{COL}\) is the lesser of the column shear above or below the joint generated by the design moments acting on the columns
3. \(d, t_f\) are the depth and flange thicknesses, respectively, of the incoming beams
4. \(V_c\) is only applicable for non-seismic application and is a reduction allowance in unbalanced shear force on the panel zone due to gravity actions determined by a rational design procedure
5. For category 1, 2 and 3 members
   a. \(M_L = M_R = C_2 M_s\)
   b. \(C_2 = 1.15\) for category 1 primary members framing into the connection
   c. \(C_2 = 1.1\) for category 2 primary members framing into the connection
   d. \(C_2 = 1.0\) for category 3 primary members framing into the connection

The \(C_2\) values are lower than overstrength values given in table 12.2.8(1) of NZS3404 (SNZ, 2007). This is because the ductility of column panel zone is greater than ductility of bolts and flanges. See discussion below.

The latest amendment to NZS3404 now includes a factor to increase \(C_2\) due to the presence of the concrete slab. In moment resisting frames where the beams support a concrete floor slab and this slab is cast against the steel columns, (MacRae et al, 2007) propose that the presence of the slab increases the flexural overstrength of the beams. The reason this occurs is due to the concrete slab being both structurally bonded to the beams and cast against the column faces. When the column starts to rotate relative to the beam, the concrete against the column face goes into compression and this is resisted by a balancing tension force through the steel beams, as shown in figure 7. When the beams framing into the column are elastic, the load path through the beam into the column is so stiff that the slab participation is negligible. However, once the beam becomes plastic, the softening of the load path through the steel means the load path through the concrete becomes increasingly significant and the overstrength moments on the columns increase. In other words slab participation diminishes with decreasing ductility demand, becoming negligible for nominal ductile or elastically responding systems or for determination of upper limit actions for more ductile systems.

![Figure 7: Slab Participation Actions (MacRae et al, 2007)](image)

It is proposed that a simplification of the provisions in NZS3404 can be made.

\[
V_p^* = \frac{M^*_L}{(d - t_f)_L} + \frac{M^*_R}{(d - t_f)_R} - V_{Col}
\]

where:

\(M^*_C = 0.9 \phi_{b,ave} Z_{ef} f_y\) for categories 1, 2 or 3

**Step 10: Calculate panel zone shear capacity and detail doubler plate**

Shear capacity of connection panel zone is set out in cl 12.9.5.3.2 of NZS3404 (SNZ, 2007) and the equation given is:
\[ \phi V_c = 0.6\phi f'_{yp} d_c \left( \frac{1 + \frac{3b_c t_p^2}{d_c d_c}}{1 + \frac{3b_c t_p^2}{d_c d_c}} \right) \]

where
\[ \phi = 0.9 \]
\[ \eta = \sqrt{1.15 - \left( \frac{N^*}{\phi N_s} \right)} \leq 1.0 \]
\[ t_p = \text{total thickness of doubler plate(s)} \]
\[ N^*/\phi N_s = \text{ratio of column design compression force to design section capacity} \]
\[ f'_{yp} = \text{design yield stress for joint panel zone} \]
\[ f'_{yp} = \frac{t_w c f_y c + t_p f_{yp}}{t_w c + t_p} \]
\[ f_{yp} = \text{yield stress of doubler plates(s)} \]
\[ d_c, b_c = \text{depth, breadth of column} \]

The column panel zone has a shear strength greater than the von Mises shear yield value on the web due to:

i) Considerable strain-hardening in shear

ii) Flexural resistance of the column flanges during panel yielding in shear

This strength is assumed to be attained at a shear distortion equal to four times the yield shear distortion. This amount of panel zone yielding may be tolerable provided that plastic hinging first develops in the beams.

The term \( \eta \), which accounts for the interaction of axial and shear force within the panel zone, does not decrease from 1.0 until \( N^*/\phi N_s \) exceeds 0.4. Experimental testing of panel zones in columns with varying ratios of \( N^*/\phi N_s \) indicate that low ratios of applied axial force to nominal section capacity have no effect on panel zone behaviour and that the standard interaction equation for shear and axial loading is conservative to apply because of the relatively rapid strain hardening rate of the panel zone (compared to the members framing into it) and its confined nature. This is allowed for in the term \( \eta \) by replacing the 1.0 value, which applies in the standard interaction equation, with 1.15.

The standard interaction equation is typically stated for the combination of tension and shear actions and may not be applicable for the interaction of compression and shear actions. In most instances \( \eta \) will be 1.0 and therefore the design procedure may be simplified by ignoring the term \( \eta \) For interior columns of internal MRFs the \( \eta \) term should be included.

For columns of non-seismic resisting systems it is unlikely that a doubler plate will be required. For columns of seismic resisting systems with one beam rigidly connected into the column about the major axis, it also unlikely that a doubler plate will be required. Doubler plates may be required for columns of seismic resisting system with two beams rigidly connected into the column about the major axis.

Detailing of doubler plates is based on the presence of continuity stiffeners. Doubler plates are sized to fit within the column web panel zone bounded by continuity stiffeners and column flanges as per figure 8.
For heavy hot rolled I section where welding is not permitted in the k area, doubler plates are detailed as per figure 9. As the doubler plate(s) are not bounded by the column flanges the doubler plate shear design capacity is based on the elastic shear capacity.

Figure 8: Doubler Plate Details, Plan view, End View

Figure 9: Doubler Plate Details for Heavy Sections, Plan

Slenderness limits are placed on panel zone elements such that the panel zone can develop its shear yield capacity. The doubler plate slenderness limit is

\[
\left( \frac{p_{dp}}{t_{dp}} \right) \left( \frac{f_{yp}}{250} \right) \leq 82
\]

(17)

The overall panel zone slenderness limit is

\[
\left( \frac{d_c - 2t_{tc}}{t_{wc} + k_1 n_{dp} t_{dp}} \right) \left( \frac{f_{yp}^*}{250} \right) \leq 82
\]

(18)

where:

- \( k_1 = 0.25 \) if the doubler plate is not plug welded to the column web
- \( k_1 = 1.0 \) if the doubler plate is plug welded to the column
- \( n_{dp} \) = number of doubler plates

Wherever possible plug welds are to be avoided. This adds to the fabrication costs.

Software Development

SCNZ is developing software to assist the steel designer in checking the requirements of the column side of the moment end plate connection. Users will be able to select beam and column details from a library of available sections. The standard beam moment end plate details will also be able to be selected. The software is being developed assuming that continuity stiffeners on the column are always required. The user will be able to select the details for the continuity stiffeners, doubler plates and backing plates. Various checks as described in this article will be made. This will greatly improve the efficiencies of designing for the user.
Design Example

Two 610UB101 Grade 300 beams are connected to a 914UB201 Grade S275 column. Part of a Category 2 MRF.

Loadings are as follows:

\[ M_1^* = 730 \text{ kNm} \quad M_2^* = 783 \text{ kNm} \]
\[ V_1^* = 350 \text{ kN} \quad V_2^* = 105 \text{ kN} \]
\[ N_{col} = 2000 \text{ kN} \]

Beam end connection is MEP-G Category 2. Details are:

M36 bolts

\[ s_g = 140 \text{ mm} \quad p_f = 90 \text{ mm} \quad s_p = 90 \text{ mm} \quad a_f = 55 \text{ mm} \quad t_i = 25 \text{ mm} \]

610UB101

\[ d = 602 \text{ mm} \quad b_f = 228 \text{ mm} \quad t_f = 14.8 \text{ mm} \quad t_w = 10.6 \text{ mm} \quad f_{yt} = 300 \text{ MPa} \]
\[ f_{yw} = 320 \text{ MPa} \]

914UB201

\[ c_d = 903 \text{ mm} \quad b_d = 303 \text{ mm} \quad t_f = 20.2 \text{ mm} \quad t_w = 15.1 \text{ mm} \quad f_{yd} = 265 \text{ MPa} \]
\[ f_{yw} = 320 \text{ MPa} \quad r_c = 19.1 \text{ mm} \]

Step 1 Size continuity stiffeners & continuity stiffener welds

Sized to be equivalent to beam flange area

Minimum stiffener width = \( (227.6-15.1)/2 = 106 \text{ mm} \)
Maximum stiffener width = \( (303 - 15.1)/2 = 144 \text{ mm} \)
Select standard flat width = 130 mm
Stiffener area required = \( (3368-14.8*15.1)(300/250) = 3773 \text{ mm}^2 \)
Required stiffener thickness = \( 3773/(2*130)=14.5 \text{ mm} \)
Select standard thickness = 16 mm
Check outstand 130/16 < 8 OK!

The column flange thickness is slightly greater than 20 mm and if this design procedure is strictly followed the continuity stiffener are required to be clipped to avoid welding in the k area. However in this design example this has not been done.

Weld to column flange and web to have capacity greater than design tension capacity of continuity stiffeners

\[ \nu_{wcf}^* = 0.9*130*16*250/(2*(130) = 1.8 \text{ kN/mm} \]
12mm FW E48xx SP \( \Phi_{w} = 1.96 \text{ kN/mm} \)

\[ \nu_{wcw}^* = 0.9*130*16*250/(824.4) = 0.57 \text{ kN/mm} \]
5mm FW E48xx SP \( \Phi_{w} = 0.82 \text{ kN/mm} \)
Step 2 Determine if Backing Plates required

The column flange thickness is less than beam end plate thickness and so backing plates required. For bolt row 2 and 3 a full depth backing plate between the continuity stiffeners is used.

Try backing plate thickness:

- \( t_{bp1} = 10\text{mm} \)
- \( t_{bp2} = t_{bp3} = 25\text{mm} \)

Step 3 Determine column flange tension effective tee stub length for each bolt row

Top bolt row

Circular yielding pattern

- \( l_{elc} = 2m_4 = 296\text{mm} \)
- \( m_4c = \frac{s_g}{2} - \frac{t_{wc}}{2} - 0.8r_c = 47.17\text{mm} \)

Side yielding

- \( l_{el2c} = 4m_4 + 1.25e_c = 291\text{mm} \)
- \( e_c = \frac{t_{wc} - s_g}{2} = 81.5\text{mm} \)

Side yielding near stiffener

- \( \lambda_{1c} = \frac{m_{4c}}{m_{4c} + e_c} = 0.37 \)
- \( \lambda_{2c} = \frac{m_{3c}}{m_{4c} + e_c} = 0.35 \)
- \( m_{3c} = a_t - 0.8t_{wscf} = 45.4\text{mm} \)

Stiffened column flange factor where \( \lambda_1 \leq 0.75 \) and \( \lambda_2 < 0.45 \)

\[
\alpha_{col} = \left\{ \frac{1.25 + 39.33\lambda_1 - 3.58\lambda_2 - 55.94\lambda_1^2 + 40.54\lambda_2^2 - 55.34\lambda_1\lambda_2 + 21.05\lambda_1^3 - 33.00\lambda_2^3}{2\pi} \right\}_{\text{min}}
\]

\( \therefore \alpha_{col} = 2\pi \)

- \( l_{e3c} = \alpha_{col}m_4 = 296\text{mm} \)

Resultant

- \( l_{ertc} = \left\{ \frac{l_{elc}}{2} \right\}_{\text{max}} \left\{ \frac{l_{e3c}}{2} \right\}_{\text{max}} + t_{bp} = 306\text{mm} \)
- \( l_{ertbp} = \left\{ \frac{l_{elc}}{2} \right\}_{\text{max}} \left\{ \frac{l_{e3c}}{2} \right\}_{\text{max}} = 296\text{mm} \)

Second bolt row

Circular yielding pattern

- \( l_{elc} = 2m_{1c} = 296\text{mm} \)
- \( m_{1c} = \frac{s_g}{2} - \frac{t_{wc}}{2} - 0.8r_c = 47.17\text{mm} \)

Side yielding

- \( l_{el2c} = 4m_{1c} + 1.25e_c = 291\text{mm} \)

Side yielding near stiffener

- \( \lambda_{1c} = \frac{m_{1c}}{m_{1c} + e_c} = 0.37 \)
- \( \lambda_{2c} = \frac{m_{2c}}{m_{1c} + e_c} = 0.50 \)
- \( m_{2c} = a_t - t_s - 0.8t_{wscf} = 64.4\text{mm} \)

Stiffened column flange factor where \( \lambda_1 \leq 0.75 \) and \( \lambda_2 \geq 0.45 \)
\[
\alpha_{col} = \left\{ \begin{array}{l}
8.13 + 4.49\lambda_1 - 3.44\lambda_2 - 16.7\lambda_1^2 + 4.66\lambda_2^2 - 6.8\lambda_1\lambda_2 + 8.75\lambda_1^3 - 1.23\lambda_2^3 \\
- 1.23\lambda_1\lambda_2^2 + 8.32\lambda_2^3 \lambda_1, \\
2\pi
\end{array} \right. \\
\text{min}
\]

\[\therefore \alpha_{col} = 2\pi\]

\[l_{e3c} = \alpha_{col} m_{t_c} = 296\text{mm}\]

Resultant

\[l_{e2c} = \left\{ l_{e2c}, \frac{l_{e2c}}{\text{max}}, l_{e2c} \langle (0.5l_{e2c} / l_{e3c} - 0.5l_{e2c}) \rangle_{\text{max}} + 0.5s_p \text{aln} + t_{bp} = 220.5\text{mm} \]

\[l_{e2bp} = \left\{ l_{e2c}, \frac{l_{e2c}}{\text{max}}, l_{e2c} \langle (0.5l_{e2c} / l_{e3c} - 0.5l_{e2c}) \rangle_{\text{max}} + 0.5s_p \text{aln} = 195.5\text{mm} \]

Third bolt row

Circular yielding pattern

\[l_{e3c} = 2nm_c = 296\text{mm}\]

\[m_c = m_{t_c}\]

Side yielding

\[l_{e2c} = 4m_c + 1.25e_c = 291\text{mm}\]

Resultant

\[l_{e3c} = \left\{ l_{e3c}, l_{e2c}, 0.5\left(e_c + s_p \text{aln}\right) \right\} = 191\text{mm}\]

\[l_{e3bp} = \left\{ l_{e2c}, 0.5\left(e_c + s_p \text{aln}\right) = 191\text{mm} \]

Step 4 Calculate tension resistance of the column flange for each bolt row

First bolt row

Mode 1

\[\phi N_{t_c} = \frac{\phi_s f_{y} l_{e2c} t_c^2}{m_c} + 0.5\phi_s f_{yb} l_{e2c} t_{bp}^2 = 705\text{KN} \]

\[m_c = m_{t_c} = 47.17\text{mm}\]

Mode 2

\[\phi N_{2c} = \frac{0.5\phi_s f_{y} l_{e2c} t_c^2 + n_c 2\phi_t N_t}{m_c + n_c} = 742\text{KN} \]

\[n_c = \left\{ 0.5e_c, 1.25m_c \text{aln} \right\} \]

\[n_c = \left\{ 0.815, 1.25 \times 47.17 \text{aln} = 59\text{mm} \]

Mode 3

\[= 2*541 = 1082\text{KN} \]

Mode 3 is not critical OK

Second bolt row

Mode 1

\[= 0.9*265*220.5*20.2^2/47.17 + 0.5*0.9*250*195.5*25^2/47.17 \]

\[= 455 + 291 \]

\[= 746\text{ kN} \]

Mode 2

\[n_c = \min(80, 81.5, 1.25*47.17) = 59\text{ mm} \]

\[=(0.5*0.9*265/1000*220.5*20.2^2 + 59*1082)/(47.17+59) \]

\[= 702\text{ kN} \]

Mode 3

\[= 2*541 = 1082\text{KN} \]

Mode 3 is not critical OK

Third bolt row

Mode 1

\[= 0.9*265*191*20.2^2/47.17 + 0.5*0.9*250*191*25^2/47.17 \]

\[= 393 + 285 \]

\[= 678\text{ kN} \]

Mode 2
\[ n_c = \min(80, 81.5, 1.25 \times 47.17) = 59 \text{ mm} \]
\[ = (0.5 \times 0.9 \times 265 + 191 \times 20.2^2 + 91 \times 1082)/(47.17 + 59) \]
\[ = 689 \text{ kN} \]
Mode 3
\[ = 2 \times 541 = 1082 \text{ kN} \]
Mode 3 is not critical OK

**Step 5 Determine column web tension capacity for bolt row 3**
Calculate effective length
\[ l_3 = 90/2 + 0.9 \times 140 = 171 \text{ mm} \]
Calculate column web tension capacity
\[ \Phi N_{3cw} = 0.9 \times 171 \times 15.1 \times 275 = 639 \text{ kN} \]

**Step 6 Determine Moment Capacity**
\[ \phi M_t = \min(N_{1r} \phi N_{1c}, \min(d_1 + \min(N_{2r}, \phi N_{2c}, d_2 + \min(N_{3r}, \phi N_{3c}, \phi N_{3w}, \phi N_{3cw}, d_3 \psi_{r3}) \right) \]
\[ = \min(794, 705) \times 650.2 + \min(774, 702) \times 505.2 + 1.0 \times \min(703, 689, 522, 639) \times 415.3 \]
\[ = 1030 \text{ kNm} \]
\[ \phi N_{3r} \phi N_{3c} \phi N_{3w} d_3 \psi_{r3} \]

Design overstrength moment action
\[ M^* = \phi \text{oms} f_{yt} \]
\[ = 1.15 \times 2900 \times 300/1000 \]
\[ = 1000.5 \text{ kNm} \]
OK!

**Step 7 Determine flange backing plate length and width**
\[ b_{bp} > 303/2 - 20.2/2 = 144 \text{ mm} \]
Use \[ b_{bp} = 145 \text{ mm} \]

Row 1 backing plate
\[ a_{ebp1} \geq [2d_r, 0.25l_{ertbp}, a_{elc}]_{\max} \]
\[ a_{ebp1} \geq [2 \times 36, 0.25 \times 296]_{\max} = 74 \text{ mm} \]
Length >\(55 - 12\)+74 = 117 mm

The row 2 and 3 backing plates are sized to fit between the continuity stiffeners.

**Step 8 Determine bolt bearing capacity into the column flange**
\[ \psi V_b = n_{bd} \phi_b \times 3.2d_f t_c f_{uc} \]
\[ = 6 \times 0.8 \times 3.2 \times 36 \times 20.2 \times 410 \]
\[ = 4580 \text{ kN} \]

**Step 9 Calculate column panel zone design shear force**
\[ V_p^* = \frac{M^*_t}{(d - t_f)_L} + \frac{M^*_c}{(d - t_f)_R} - V_{col} \]
\[ M^*_t = M^*_C = C_\text{d} M_{ax} = 1.1 \times 782/0.9 = 956 \text{ kNm} \]
\[ V_{col} = 440 \text{ kN (assumed)} \]
\[ V_p^* = 956/(602.6 - 14.8) + 956/(602.6 - 14.8) - 440 = 2810 \text{ kN} \]

**Step 10 Determine column panel zone capacity**
Without a doubler plate
\[ \psi V_c = 0.6 \phi f_{yc} d_c \left( \psi_{wc} + n_{dp} t_{dp} \right) \left( 1 + \frac{3b_c t_{fc}^2}{d_c \psi_{wc} + n_{dp} t_{dp}} \right) \]
\[ = 0.6 \times 275 \times 0.9 \times 903 \times 15.1 \times (1 + (3 \times 303 \times 20.2^2)/(602.6 \times 903 \times 15.1)) = 2116 \text{ kN} \]
Require a doubler plate

Column flange is of a size where strictly speaking welding in the k area should be avoided. In this example this has not been done.

Try 6mm doubler plate

\[
f_{yd}^* = \frac{f_{yc} + n_{dp} f_{yd}}{t_{wc} + n_{dp} t_{dp}}
\]

\[
f_{yd}^* = \frac{15.1 \times 275 + 1 \times 6 \times 260}{15.1 + 1 \times 6} = 271
\]

\[
\phi V_c = 0.6 \frac{f_{yd}^* r_c}{t_{wc} + n_{dp} t_{dp}} \left[ 1 + \frac{3 b_c t_{fc}^2}{4 d_c (t_{wc} + n_{dp} t_{dp})} \right]
\]

\[
\phi V_c = 0.6 \times 0.9 \times 271 \times 903 \times (5.1 + 1 \times 6) \left[ 1 + \frac{3 \times 303 \times 20.2^2}{602.6 \times 903 \times (5.1 + 1 \times 6)} \right] = 2878 \text{kN}
\]

Doubler plate slenderness must satisfy

\[
\frac{b_{dp}}{t_{dp}} \left( \frac{f_{yd}}{250} \right) \leq 82
\]

\[
\frac{824}{6} \left( \frac{280}{250} \right) = 145 > 82
\]

Therefore plug weld is required.

If do not want plug weld than a 10mm plate is required to satisfy slenderness limit.

Dimensions of the doubler plate

Depth = 602.6 - 2*16 - 2*(6 + 2) = 554.6 \approx 554

Width = 903 - 2*20.2 - 2*19.1 = 824.4 \approx 824 \text{mm}
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Appendix – Proposed Formula to be included in SCNZ 14-1:2007

Design Formulae: Column Flange

**Governing Criteria**

\[ V^* \leq \phi V_r \]
\[ M^* \leq \phi M_r \]
\[ M'_{oms} \leq \phi M_r \]

- \( b_d \) ≥ \( s_g + 3.0d_r \)
- \( t_{bpx} \leq 1.5t_{fc} \)

\[ b_{bp} = \frac{b_{fc}}{2} - \frac{t_{wc}}{2} \]

- \( a_{ebp1} \geq [2d_r,0.25r_1er1bp,a_{elc}]_{\text{max}} \)
- \( a_{ebp2} \geq [2d_r,0.25r_2er2bp]_{\text{max}} \)
- \( a_{ebp3} \geq [2d_r,0.5r_3er3bp]_{\text{max}} \)

**Design Actions**

\[ M^* = \phi_{oms}Z_{ex}f_{yf} \]
\[ V^* = \phi_{oms}f_{yf} \]

**Connection Design Strength Limits**

\[ \phi M_r = \left[ N_{1c},\phi N_{2c} \right]_{\text{min}} + \left[ N_{2c},\phi N_{3c} \right]_{\text{min}} + \left[ N_{3c},\phi N_{3cw} \right]_{\text{min}}d_3 \]

\[ \phi V_r = \left[ V_{con},\phi V_{bfc} \right]_{\text{min}} \]

**Column Flange Bolt Row Design Tension Capacities**

**General**

- \( \phi N_{xc} = \left[ N_{1c},\phi N_{2c},\phi N_{3c} \right]_{\text{min}} \)

- \( \phi N_{1c} = \frac{f_yd_{x}x_{c}+0.5f_yb_{x}y_{b}x_{bpx}t_{bpx}}{m_c} \)

- \( \phi N_{2c} = \frac{0.5f_yd_{x}x_{c}+2n_c \phi N_{ef}}{m_c + n_c} \)

- \( \phi N_{3} = 2\phi b_{ef} \)

- \( e_c = \frac{f_{ec} - s_g}{2} \)

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Effective T-Stub Length: Top Row of Bolts with Stiffener

\[ m_{3c} = a_t - 0.8t_{\text{wscf}} \]
\[ m_{4c} = \frac{s_g}{2} - \frac{t_{\text{wscf}}}{2} - 0.8r_c \]
\[ m_c = m_{4c} \]
\[ n_c = \left\{ e_c, 1.25m_c \right\}_{\text{min}} \]
\[ \lambda_{1c} = \frac{m_{4c}}{m_c + e_c} \]
\[ \lambda_{2c} = \frac{m_{3c}}{m_c + e_c} \]
\[ l_{er1c} = \left\{ e_c, e_{2c}, l_{\text{e3c}} \right\}_{\text{max}} \left\{ e_c, l_{\text{e3c}} \right\}_{\text{max}} + t_{bp} \]
\[ l_{er1bp} = \left\{ e_c, e_{2c}, l_{\text{e3c}} \right\}_{\text{max}} \left\{ e_c, l_{\text{e3c}} \right\}_{\text{max}} + t_{bp} \]
\[ l_{etc} = 2\pi m_4 \]
\[ l_{e2c} = 4m_4 + 1.25e_c \]
\[ l_{e3c} = \alpha_{\text{co}} m_4 \]
\[ l_{e5c} = 2m_4 + 0.625e_c + a_{\text{etc}} \]
\[ l_{e6c} = \alpha_{\text{co}} m_4 - (2m_4 + 0.625e_c) + a_{\text{etc}} \]

Bolt distance from stiffener weld
Bolt distance from column web root
Effective edge distance column flange
Edge distance ratios
Top bolt row effective T-stub length column flange
Top bolt row effective T-stub length backing plate
Circular yielding pattern
Side yielding pattern
Side yielding near stiffener pattern
Corner yielding
Corner yielding near a stiffener

Effective T-Stub Length: Second Row of Bolts with Stiffener

\[ m_{1c} = \frac{s_g}{2} - \frac{t_{\text{wscf}}}{2} - 0.8r_c \]
\[ m_{2c} = p_t - t_s - 0.8t_{\text{wscf}} \]
\[ m_c = m_{1c} \]
\[ n_c = \left\{ e_c, 1.25m_c \right\}_{\text{min}} \]
\[ \lambda_{1c} = \frac{m_{1c}}{m_c + e_c} \]
\[ \lambda_{2c} = \frac{m_{2c}}{m_c + e_c} \]
\[ l_{er2c} = \left\{ e_c, e_{2c}, l_{\text{e3c}} \right\}_{\text{max}} \left\{ e_c, l_{\text{e3c}} \right\}_{\text{max}} + t_{bp} \]
\[ l_{er2bp} = \left\{ e_c, e_{2c}, l_{\text{e3c}} \right\}_{\text{max}} \left\{ e_c, l_{\text{e3c}} \right\}_{\text{max}} + t_{bp} \]
\[ l_{et2c} = \left\{ e_c, e_{2c}, l_{\text{e3c}} \right\}_{\text{max}} \left\{ e_c, l_{\text{e3c}} \right\}_{\text{max}} + t_{bp} \]

Bolt distance from column web root
Bolt distance from stiffener weld
Effective edge distance column flange
Edge distance ratios
Second of two rows column flange
Second of two rows column flange backing plate
Second of three rows column flange

1 SCI, "Joints in Steel Construction: Moment Connections", P207/95, Steel Construction Institute, UK, 1995, pp.23, 139
2 SCI, "Joints in Steel Construction: Moment Connections", P207/95, Steel Construction Institute, UK, 1995, pp.23, 139
Second of three rows column flange backing plate

\[ l_{2bp} = \left[ l_{2c} \left( l_{2c} \max \right) + l_{1c} \left( 0.5 l_{2c} \left( l_{3e} - 0.5 l_{2c} \right) \right) \right] + 0.5 s_p \]

Circular yielding pattern

\[ l_{1e1c} = 2 \pi m_1c \]

Side yielding pattern

\[ l_{e2c} = 4 m_1c + 1.25 e_c \]

Side yielding near stiffener pattern

**Effective T-Stub Length: Third Row of Bolts**

\[ m_c = m_1c \]

Effective edge distance column flange

\[ n_c = \left[ l_{e1c} \left( 1.25 m_1 \right) \right] \]

Third bolt row effective T-stub length column flange

\[ l_{e3c} = \left[ l_{e1c} l_{e2c} \left( 0.5 l_{e2c} + s_p \right) \right] \]

Third bolt row effective T-stub length column flange backing plate

\[ l_{2bp} = \left[ l_{3c} l_{e2c} \left( 0.5 l_{2c} + s_p \right) \right] \]

Circular yielding pattern

\[ l_{e1c} = 2 \pi m_c \]

Side yielding pattern

\[ l_{e2c} = 4 m_c + 1.25 e_c \]

**Column Web Tension Capacity**

\[ \phi N_{3ow} = \phi_3 l_3 t_{wc} f_{cyw} \]

Third bolt row

\[ l_3 = \left( 0.9 s_g \frac{s_p}{2} \right) + 0.9 s_g \]

Effective tensile column web length 3rd row

**Column Flange Bolt Bearing Capacity**

\[ \phi V_{bc} = n_{bc} \phi_s 3.2 f_{wc} d_f \]

Column flange bolt bearing

**Definitions of Terms**

\[ a_{e1c} \quad \text{Bolt edge end distance column flange} \]

\[ a_{ebp1} \quad \text{Bolt edge end distance backing plate bolt row 1} \]

\[ a_{ebp2} \quad \text{Bolt edge end distance backing plate bolt row 2} \]

\[ a_{ebp3} \quad \text{Bolt edge end distance backing plate bolt row 3} \]

**Design Formulae: Continuity Stiffeners**

**Governing Criteria**

\[ \frac{b_f - t_{wc}}{2} \leq b_s \leq \frac{b_h - t_{wc}}{2} \]

Continuity stiffener width

\[ A_{spar} \geq \phi \left( t_f - t_{wc} \right) \left( \frac{f_{ys}}{f_{ys}} \right) \]

Continuity stiffener area – pair

\[ A_{spar} \geq \phi \left( t_f - t_{wc} \right) \left( \frac{f_{ys}}{f_{ys}} \right) \]

Continuity stiffener area – pair
Continuity stiffener thickness

Design Formulae: Column Web Panel

### Governing Criteria

- \[ V_p^* \leq \phi V_c \]  
  - Column panel zone horizontal shear

- \[ \left( \frac{d_p - 2t_{kc}}{f_{yd}} \right) \leq 82 \]  
  - Slenderness limit for doubler plate

- \[ \left( \frac{d_c + 0.25n_{dp}t_{dp}}{f_{yd}} \right) \leq 82 \]  
  - Slenderness limits on overall panel zone, no plug weld

- \[ 6 \leq t_p \leq \frac{t_n}{16} \]  
  - Doubler plate thickness

### Design Actions

- \[ V_p^* = \frac{M_L^*}{(d - t_f)_L} + \frac{M_R^*}{(d - t_f)_R} - V_{Col} \]  
  - Design shear action for joint panel zone - seismic actions

- \[ V_p^* = \frac{M_L^*}{(d - t_f)_L} + \frac{M_R^*}{(d - t_f)_R} - V_{Col} - V_G \]  
  - Design shear action for joint panel zone – gravity only actions

- \[ M_L^* \]  
  - Design moment: Elastic - Left hand beam

- \[ M_R^* \]  
  - Design moment: Elastic - Right hand beam

- \[ M_{col} \]  
  - Design moment: Category 1, 2 and 3 – Left hand beam

- \[ M_{col} \]  
  - Design moment: Category 1, 2 and 3 – Right hand beam

- \[ M_{col} \]  
  - Is the lesser of the column shear above or below the joint generated by the design moments acting on the columns

- \[ V_G \]  
  - Unbalanced shear force on panel zone due to gravity action in connections supporting beams from 2 opposing directions

### Connection Design Strength Limits

\[ \phi V_c = 0.6 \phi_{yd}d_c (c_w + n_{dp}t_{dp}) \left[ 1 + \frac{3b_c t_c^2}{dd_c (c_w + n_{dp}t_{dp})} \right] \]  

- Shear capacity of column panel zone, HW and HR \( t_c \leq 32 \text{mm} \)
\[ \phi V_c = 0.6\phi f_{ycw} d_c t_{wc} \left[ 1 + \frac{3b_d t_{dp}^2}{d_b d_c n_{dp} t_{dp}} \right] + 0.5\phi f_{ydp} b_{dp} n_{dp} t_{dp} \]

Shear capacity of column panel zone, \( t_{fc} > 32 \text{mm}, \) doubler plate not welded to column flanges

**Definitions of Terms**

\[ f_{yp} = \frac{t_{wc} f_{ycw} + n_{dp} t_{dp} f_{ydp}}{t_{wc} + n_{dp} t_{dp}} \]

Design yield stress for joint panel zone

\[ n_{dp} \]

Number of doubler plates, 1 or 2

\[ b_{dp} = d_c - 2t_{fc} - 2t_{cw} \]

Doubler plate width HW column

\[ b_{dp} = d_c - 2t_{fc} - 2r_{fc} \]

Doubler plate width HR column, \( t_w \leq 32 \text{mm} \)

\[ b_{dp} = d_c - 2t_{fc} - 2r_{fc} - 2t_{dp} - 70 \text{mm} \]

Doubler plate width HR column, \( t_w > 32 \text{mm} \)

\[ d_{dp} = d_b - 2t_s - 2t_{wscw} \]

Doubler plate depth