NCCI: Design of portal frame eaves connections

This NCCI provides information on the design method for a bolted eaves moment connection. It includes several simplifications which are explained throughout the document, to obtain simpler but conservative calculations.

Contents

1. Design model 2
2. Parameters 4
3. Weld design 6
4. Potential resistances of bolt rows in the tension zone 7
5. Assessment of the compression zone 12
6. Column web panel in shear 14
7. Rafter web in compression 15
8. Force distribution in bolt rows 15
9. Assessment of the shear resistance 19
10. Limits of application 20
11. Background. 21
1. Design model

1.1 Stiffness

According to §5.2.1(1) of EN 1993-1-8, a joint can be classified as rigid, nominally pinned or semi-rigid according to its rotational stiffness by comparing its initial stiffness, $S_{j,ini}$ with the classification boundaries given in §5.2.2.5 of EN 1993-1-8. The initial stiffness of a joint connecting H or I sections may be calculated according to the rules given in §6.3.1 of EN 1993-1-8.

A joint may be classified on the basis of experimental evidence, experience of previous satisfactory performance in similar cases or by calculations based on test evidence.

1.2 Strength

1.2.1 General model

The moment resistance, $M_{j,Rd}$, and the shear resistance, $V_{j,Rd}$, of the joint depend on the connected members and the basic components of the joint that make a contribution to the joint resistance: bolts, column web and flanges, haunch and rafter web and flanges and welds, see Figure 1.1. §6.1.3 and Table 6.1 of EN 1993-1-8 provide the information to identify the basic joint components.

---

**Figure 1.1**  Portal frame eaves connections with bolted end plate

**Key:**
- 1. Column
- 2. Eaves haunch
- 3. Rafter
- 4. Flange Weld
- 5. Web weld
- 6. Bolts
- 7. End plate
- 8. Shear bolts
- 9. Optional tension stiffeners
- 10. Compression stiffener

A. Tension zone  B. Shear zone  C. Compression zone
Some countries treat the bolts in the tension zone and in the shear zone as one group of bolts, therefore, extra bolts (noted * in Figure 1.1) may be required to satisfy the spacing requirements in Table 3.3 of EN 1993-1-8. Other countries treat them as two separate bolt groups, and no additional bolts are necessary.

According to EN 1993-1-8 §6.2.7.2, once the basic components have been identified, the design moment resistance of the eaves bolted end-plate joints may be determined from:

\[ M_{j,Rd} = \sum_r h_r F_{tr,Rd} \]

where:
- \( F_{tr,Rd} \) is the effective design tension resistance of bolt-row \( r \),
- \( h_r \) is the distance from bolt-row \( r \) to the centre of compression; this can be taken as the middle of the compression flange of the haunch.
- \( r \) is the bolt-row number.

The joint must satisfy:

\[ \frac{M_{j,Ed}}{M_{j,Rd}} \leq 1,0 \]

The procedure to determine the joint resistance is presented in Table 1.1.

### Table 1.1 Procedure to determine \( F_{tr,Rd} \) and the joint resistance

<table>
<thead>
<tr>
<th>Step</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Calculate potential tension resistance of each bolt row in the tension zone</td>
<td>( F_{t,Rd}^{(row)} )</td>
</tr>
<tr>
<td>2. Calculate the design compression resistance in the compression zone</td>
<td>( F_{c,Rd} )</td>
</tr>
<tr>
<td>3. Calculate the design shear resistance of the column web panel</td>
<td>( V_{wp,Rd} )</td>
</tr>
<tr>
<td>4. Calculate the effective design tension resistance of each bolt row</td>
<td>( F_{tr,Rd} )</td>
</tr>
<tr>
<td>5. Calculate the moment resistance of the joint</td>
<td>( M_{j,Rd} = \sum_r h_r F_{tr,Rd} )</td>
</tr>
<tr>
<td>6. Assessment for vertical shear forces</td>
<td>( V_{Ed} \leq V_{Rd} )</td>
</tr>
</tbody>
</table>

### 1.2.2 Simplifications

Several simplifications have been done in this NCCI in order to make the calculation of the eaves moment connections easier, leading to a conservative approach. These are described below:

- In the full calculation the tying resistance of bolt rows should be calculated by considering bolt rows individually and bolt rows as part of groups of bolt rows, and taking the minimum resistance obtained. In this simple approach, only the individual bolt rows are considered. This leads to conservative results but saves a lot of time and effort in the process.
The effective length of each T-stub to calculate the tying resistance of the bolt row is taken as the minimum possible effective length to avoid the superposition of the effective lengths of the different bolt rows. This is shown in section 4.1 in this document.

Based on §6.2.2(2) of EN 1993-1-8 the tension zone and the shear zone are treated separately. It is assumed that bolts in the tension zone support only tension and no shear. Similarly the bolts in the shear zone only support shear and no tension.

§4 of EN 1993-1-8 gives rules for weld design. Weld design is usually carried out after the calculation of the design resistance of the connection. However, this NCCI gives simple rules for the initial sizing of the welds. It specifies full strength welds, which leads to a simple calculation procedure. Further methods for weld design are given in Annex A of this NCCI.

2. Parameters

![Diagram showing parameters]

Figure 2.1 Portal frame eave: Parameter definition

- \( a \) effective throat thickness of the weld;
- \( A_{fb} \) cross section of the rafter flange
- \( b_p \) width of the end plate
- \( d_1 \) distance from the top of the tension flange of the rafter to the edge of the end plate
- \( d_2 \) pitch between the bolt row in the extended zone of the end plate and the first bolt row below the tension flange of the rafter
- \( d_3 \) distance from the last shear bolt row to the bottom of the compression flange of the haunch
- \( e_1 \) vertical distance from the edge of the column flange to the first bolt row
- \( e_c \) horizontal distance from the edge of the column flange to the bolt line
- \( e_p \) horizontal distance from the edge of the end plate to the bolt line
\( e_{pl} \)  
- distance between the bottom of the compression flange of the haunch and the edge of the end plate

\( e_s \)  
- vertical distance from the edge of the end plate to the first bolt row

\( f_{ub} \)  
- ultimate strength of the bolt

\( f_{u,b} \)  
- ultimate strength of the rafter

\( f_{u,c} \)  
- ultimate strength of the column

\( f_{u,h} \)  
- ultimate strength of the haunch

\( f_{u,p} \)  
- ultimate strength of the end plate

\( f_{y,b} \)  
- yield strength of the rafter

\( f_{y,c} \)  
- yield strength of the column

\( f_{y,h} \)  
- yield strength of the haunch

\( f_{y,p} \)  
- yield strength of the end plate

\( h_c \)  
- depth of the column

\( h_p \)  
- depth of the end plate

\( m \)  
- distance from the centre of a bolt to 20\% distance into the adjacent haunched rafter weld to the end-plate or distance from the centre of a bolt to 20\% distance into the column web root (as indicated in Figure 6.2 of EN1993-1-8).

\( n_s \)  
- number of bolts in shear

\( n_t \)  
- number of horizontal bolt rows in tension

\( p \)  
- pitch between bolt rows in the tension zone

\( p_2 \)  
- pitch between the last tension bolt and the first shear bolt

\( p_3 \)  
- pitch between bolt rows in the shear zone

\( r \)  
- is the bolt row number, the bolt-rows are numbered starting from the bolt-row furthest from the centre of compression;

\( w \)  
- gauge (i.e. distance between cross centres)

\( t_{fb} \)  
- thickness of the rafter flange

\( t_{fc} \)  
- thickness of the column flange

\( t_p \)  
- thickness of the end plate

\( t_{wb} \)  
- thickness of the rafter web

\( t_{wc} \)  
- thickness of the column web
3. **Weld design**

3.1 **Tension flange to end-plate weld**

Conservatively a full strength weld is appropriate. This requirement will be satisfied provided the weld throat thickness is such that:

\[
a \geq t_{fb} \left( \frac{f_y}{\gamma_{M0}} \left( \frac{\beta_w \gamma_{M2}}{f_u / \sqrt{2}} \right) \right)
\]

where:

- \( f_y \) is yield strength of rafter section
- \( f_u \) is nominal ultimate strength of the weaker part joined (i.e. end plate or rafter section)
- \( \beta_w \) is the correlation factor from Table 4.1 of EN 1993-1-8

When \( \gamma_{M0} = 1.0 \) and \( \gamma_{M2} = 1.25 \):

- \( a \geq 0.46 \cdot t_{fb} \) for a S235 beam
- \( a \geq 0.48 \cdot t_{fb} \) for a S275 beam
- \( a \geq 0.55 \cdot t_{fb} \) for a S355 beam

Additional calculation methods are given in Annex A of this NCCI.

3.2 **Web to end-plate weld**

Rafter web welds in the tension zone should conservatively be full strength. It is sensible to use this full strength weld for the full web depth as well.

This requirement will be satisfied provided the weld throat thickness is such that:

\[
a \geq t_{wb} \left( \frac{f_y}{\gamma_{M0}} \left( \frac{\beta_w \gamma_{M2}}{f_u / \sqrt{2}} \right) \right)
\]

where:

- \( f_y \) is yield strength of rafter section
- \( f_u \) is nominal ultimate strength of the weaker part joined (i.e. end plate or rafter section)
- \( \beta_w \) is the correlation factor from Table 4.1 of EN 1993-1-8

When \( \gamma_{M0} = 1.0 \) and \( \gamma_{M2} = 1.25 \):

- \( a \geq 0.46 \cdot t_{wb} \) for a S235 beam
\[ a \geq 0.48 \cdot t_{wb} \text{ for a S275 beam} \]
\[ a \geq 0.55 \cdot t_{wb} \text{ for a S355 beam} \]

### 3.3 Compression flange welds

If the compression flange has a properly sawn end, a nominal weld is sufficient and the following throat thicknesses are recommended:
- 5 mm fillet welds or
- 4 mm fillet welds, for beams with flange thickness of 12 mm or less

In other cases, the weld must be designed to carry the full compressive force expected in the haunch flange.

### 4. Potential resistances of bolt rows in the tension zone

**NOTE:** EN 1993-1-8 uses the symbol \( F_{t,Rd} \) to refer to both the tension resistance of an individual bolt row and the tension resistance of one bolt. In this document \( F_{t,Rd(\text{row})} \) has been used to refer to the tension resistance of the row.

For each bolt row, the potential design tension resistance is given in **EN 1993-1-8 §6.2.7.2(6):**

\[
F_{t,Rd(\text{row})} = \min \left( F_{t,fc,Rd}; F_{t,wc,Rd}; F_{t,\text{ep},Rd}; F_{t,wb,Rd} \right)
\]

#### Table 4.1 Components of the joint to determine the potential design resistance of a bolt row

<table>
<thead>
<tr>
<th>Component</th>
<th>Section number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column flange in bending</td>
<td>( F_{t,fc,Rd} ) 4.1</td>
</tr>
<tr>
<td>Column web in transverse tension</td>
<td>( F_{t,wc,Rd} ) 4.2</td>
</tr>
<tr>
<td>End-plate in bending</td>
<td>( F_{t,\text{ep},Rd} ) 4.3</td>
</tr>
<tr>
<td>Rafter web in tension</td>
<td>( F_{t,wb,Rd} ) 4.4</td>
</tr>
</tbody>
</table>

The potential design tension resistance \( F_{t,Rd(\text{row})} \) for each bolt-row should be determined in sequence, starting from the furthest bolt row from the centre of compression (bolt row 1) and then progressing to the next one (bolt-row 2) until the last one, the closest one to the centre of compression, is calculated (see Figure 4.1). Assume the centre of compression is in line with the centre of the compression flange of the haunch.
For simplicity and ease of calculations, the potential design tension resistance of each bolt-row assumes that there is no overlap with other bolt-rows.

This simplified approach leads to conservative results assuming that T-stub effective length $l_{e_{ff}}$ is determined accordingly, see worked example SX031.

The effective design tension resistance $F_{t,Rd}$ for each bolt row may be less than the potential design tension resistance $F_{t,Rd(row)}$.

4.1 Column flange in bending

The design resistance and failure mode of an unstiffened column flange in transverse bending, together with the associated bolts in tension, should be taken as similar to those of an equivalent T-stub flange.

$$F_{t,fc,Rd} = \min (F_{T,1,Rd,}, F_{T,2,Rd,}, F_{T,3,Rd,});$$ accounting for prying forces and the three failure modes (see table 4.2 below). This is the same as Table 6.2 of EN 1993-1-8 §6.2.4:

<table>
<thead>
<tr>
<th>Failure mode</th>
<th>Design resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode 1: Complete flange yielding</td>
<td>$F_{t,1,Rd} = \frac{4 \cdot M_{pl,1,Rd}}{m}$</td>
</tr>
<tr>
<td>Mode 2: Bolt failure with flange yielding</td>
<td>$F_{t,2,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \sum F_{t,Rd}}{m + n}$</td>
</tr>
<tr>
<td>Mode 3: Bolt failure</td>
<td>$F_{t,3,Rd} = \sum F_{t,Rd}$</td>
</tr>
</tbody>
</table>
where:

\[ F_{t,Rd} = \frac{0.9 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} \]

is the tension resistance of non countersunk bolts.

\[ \sum F_{t,Rd} = 2F_{t,Rd} \quad \text{i.e. two bolts per row} \]

\[ M_{pl,1,Rd} = 0.25 \sum \ell_{eff,1} t_{fc} f_y / \gamma_{M0} \]

\[ M_{pl,2,Rd} = 0.25 \sum \ell_{eff,2} t_{fc} f_y / \gamma_{M0} \]

\[ n = e_{min} \quad \text{but} \quad n \leq 1.25 m, \text{ see Figure 6.2 in EN 1993-1-8} \]

\[ \sum l_{eff} \] can be determined according to Figure 6.2, Figure 6.9 and Table 6.4 (for unstiffened columns) or Table 6.5 (for stiffened columns) of EN 1993-1-8.

Alternatively a simple conservative approach as given below can be used.

For an individual bolt row the following simplification can be made:

\[ \sum l_{eff,1} = \sum l_{eff,2} = L_{eff} \quad \text{as shown in figure 4.2 below} \]

\[ \sum l_{eff,1} \] is the value of \( \sum l_{eff} \) for mode 1

\[ \sum l_{eff,2} \] is the value of \( \sum l_{eff} \) for mode 2

This method is based on the assumption that the effective length is always limited to a maximum distance of the pitch between bolt centres. Figure 4.2 and table 4.3 illustrate this approach.
Figure 4.2   Effective lengths of the T-stub in (a) extended end plate and (b) flush end plate connections.
### Table 4.3  Effective length for each bolt row

<table>
<thead>
<tr>
<th>End bolt row</th>
<th>Inner bolt row</th>
<th>End bolt row adjacent to a stiffener (stiffened column flange) or outside the tension flange of the rafter (end plate)</th>
<th>Inner bolt row adjacent to a stiffener (stiffened column flange) or below the tension flange of the rafter (end plate)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2(\pi m)</td>
<td>2(\pi m)</td>
<td>2(\pi m)</td>
</tr>
<tr>
<td></td>
<td>(\pi m + 2e_1)</td>
<td>(\pi m + 2e_1)</td>
<td>(\pi m + p)</td>
</tr>
<tr>
<td></td>
<td>(e_1 + \alpha m - (2m + 0.625e))</td>
<td>(e_1 + \alpha m - (2m + 0.625e))</td>
<td>(0.5 + \alpha m - (2m + 0.625e))</td>
</tr>
<tr>
<td>2(\pi m)</td>
<td>(2\pi m)</td>
<td>(2\pi m)</td>
<td>(2\pi m)</td>
</tr>
<tr>
<td>(\pi m + 2e_1)</td>
<td>(2\pi m)</td>
<td>(2\pi m)</td>
<td>(2\pi m)</td>
</tr>
<tr>
<td>4(m + 1,25e)</td>
<td>4(m + 1,25e)</td>
<td>(4m + 1,25e)</td>
<td>(4m + 1,25e)</td>
</tr>
<tr>
<td>2(m + 0,625e + 0,5p)</td>
<td>(p)</td>
<td>(2m + 0,625e + 0,5p)</td>
<td>(2m + 0,625e + 0,5p)</td>
</tr>
<tr>
<td>(e_1 + 0,5p)</td>
<td>(p)</td>
<td>(e + 2m_e + 0,625e_x)</td>
<td>(e + 2m_e + 0,625e_x)</td>
</tr>
<tr>
<td></td>
<td>(0,5b_p)</td>
<td>(0,5b_p)</td>
<td>(0,5b_p)</td>
</tr>
<tr>
<td></td>
<td>(0,5w + 2m_x + 0,625e_x)</td>
<td>(0,5w + 2m_x + 0,625e_x)</td>
<td>(0,5w + 2m_x + 0,625e_x)</td>
</tr>
</tbody>
</table>

### 4.2  Column web in transverse tension

The transverse tension resistance for an unstiffened column web is given in EN 1993-1-8 §6.2.6.3 as:

\[
F_{t,\text{wc},\text{Rd}} = \frac{\omega b_{\text{eff},t,\text{wc}} t_{\text{wc}} f_{y,\text{wc}}}{\gamma_{M0}}
\]

where:

\[
\omega = \frac{1}{\sqrt{1 + 1.3(b_{\text{eff},\text{wc}} t_{\text{wc}} / A_{\text{wc}})^2}}
\]

is the reduction factor to allow for the interaction with shear in the column web panel.

\(A_{\text{wc}}\)

is the shear area of the column, see EN 1993-1-1 §6.2.6(3). For rolled I and H sections it can be conservatively taken as \(h_w t_w\).

\(b_{\text{eff},t,\text{wc}} = l_{\text{eff}}\), see section 4.1

### 4.3  End-plate in bending

The design resistance and failure mode of an end-plate in bending, together with the associated bolts in tension, can be determined following the methodology given in section 4.1 of this document for column flange in bending and using Table 6.6 instead of Table 6.4 of EN 1993-1-8.

\[
F_{t,\text{ep},\text{Rd}} = \min(F_{T,1,Rd}; F_{T,2,Rd}; F_{T,3,Rd})
\]
4.4 Rafter web in tension

The resistance of the rafter web in tension for an unstiffened web can be calculated according to EN 1993-1-8 §6.2.6 as follows:

\[ F_{t,wb,Rd} = \frac{b_{\text{eff},t,wb} f_{y,wb}}{\gamma_M} \]

where:

\[ b_{\text{eff},t,wc} = l_{\text{eff}}, \text{ see section 4.1} \]

5. Assessment of the compression zone

The design compression resistance of the compression zone may be calculated as follows:

\[ F_{c,Rd} = \min(F_{c,\text{wc},Rd}; F_{c,\text{fh},Rd}) \]

For \( F_{c,\text{wc},Rd} \) and \( F_{c,\text{fh},Rd} \) see sections 5.1 and 5.2 below.

In addition, it is necessary to assess that:

\[ F_{c,\text{Ed}} \leq F_{c,Rd} \]

Due to the fact that haunches in portal frames are typically long enough, the component of the compression force in the direction of the haunch can be considered as the horizontal component acting in the column web, which is the sum of the tensile resistances of the bolts:

\[ F_{c,\text{Ed}} = \sum F_{t,Rd(\text{row})} \]

Table 5.1 Components of the joint involved in the assessment of the compression zone

<table>
<thead>
<tr>
<th>Component</th>
<th>Section number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column web in compression</td>
<td>( F_{c,\text{wc,Rd}} ) 5.1</td>
</tr>
<tr>
<td>Haunch flange and web in compression</td>
<td>( F_{c,\text{fh,Rd}} ) 5.2</td>
</tr>
</tbody>
</table>

5.1 Column web in transverse compression

5.1.1 Requirement of a compression stiffener

A stiffener is needed when the column web in compression is not strong enough to take all the compression force. The stiffener also guards against the buckling of column web.

The stiffener also improves the stability of the column, especially if there is a plastic hinge forming at this position.

In most practical cases a compression stiffener will be required.
5.1.2 Column web with a compression stiffener

The design resistance of a stiffened column subject to transverse compression may be done in accordance with §9.1(3) of EN 1993-1-5.

5.1.3 Column web without a compression stiffener

The design resistance of an unstiffened column web subject to transverse compression is given in EN 1993-1-8, §6.2.6:

\[
F_{c,wc,Rd} = \min \left( \frac{\omega k_{wc} b_{eff,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}; \frac{\omega k_{wc} \rho b_{eff,wc} t_{wc} f_{y,wc}}{\gamma_{M1}} \right)
\]

where:

- \(\omega\) is a reduction factor, see section 4.2
- \(b_{eff,wc} = t_{fb} + 2\sqrt{2} a_p + 5(t_{fc} + s) + s_p\); is the effective width of column web in compression for bolted end-plate connections.
- \(a_p\) is the weld throat.
- \(s_p\) is the length obtained by dispersion at 45° through the end-plate (at least \(t_p\) and, provided that the length of end-plate below the flange is sufficient, up to \(2t_p\)).
- \(\rho\) is the reduction factor for plate buckling:
  - if \(\bar{\lambda}_p \leq 0.72\) then \(\rho = 1.0\)
  - if \(\bar{\lambda}_p > 0.72\) then \(\rho = \frac{(\bar{\lambda}_p - 0.2)}{\bar{\lambda}_p^2}\)

where

\[
\bar{\lambda}_p = 0.932 \sqrt{\frac{b_{eff,wc} d_{wc} f_{y,wc}}{Et_{wc}^2}}\]

it the plate slenderness.

for a rolled I or H section column:
\[d_{wc} = h_c - 2(t_{fc} + r_c)\]

for a welded I or H section column:
\[d_{wc} = h_c - 2(t_{fc} + \sqrt{2}a_c)\]

\(k_{wc}\) is a reduction factor accounting the maximum longitudinal compressive stress \(\sigma_{com,Ed}\) due to axial force and bending moment in the column web (adjacent to the root radius for a rolled section or the toe of the weld for a welded section)

- when \(\sigma_{com,Ed} \leq 0.7f_{y,wc}\) then \(k_{wc} = 1.0\)
- when \(\sigma_{com,Ed} > 0.7f_{y,wc}\) then \(k_{wc} = 1.7 - \sigma_{com,Ed}/f_{y,wc}\)
In preliminary calculations, a value of $k_{wc} = 1.0$ is recommended, as well as checking the value later, once the longitudinal stress is known. However, the value of $k_{wc} = 0.7$ can conservatively be used.

### 5.2 Haunch flange and web in compression

The compression resistance of the haunch flange is given by the following expression in §6.2.6.7 of EN 1993-1-8.\[ F_{c,fl,Rd} = \frac{M_{c,Rd}}{(h - t_{fh})} \]

- $h$ is the depth of the beam including rafter and haunch.
- $M_{c,Rd}$ is the design moment resistance of the beam (rafter + haunch) cross-section, reduced if necessary to allow for shear, see EN 1993-1-1 §6.2.5. $M_{c,Rd}$ may be calculated neglecting the intermediate flange.
- $t_{fh}$ is the flange thickness of the connected haunch.

If the height of the beam (rafter + haunch) exceeds 600 mm the contribution of the rafter web to the design compression resistance should be limited to 20%. This means that if the resistance of the flange is $t_{fh}b_{fl}f_{y,fl}$ then:

\[ F_{c,fl,Rd} \leq \frac{t_{fh}b_{fl}f_{y,fl}}{0.8} \]

### 6. Column web panel in shear

Provided the column web slenderness satisfies the following condition: \[ \frac{d}{t_w} \leq 69 \gamma_c \], the resistance of the column web panel in shear for an unstiffened column, according to §6.2.6.1 of EN 1993-1-8, is:

\[ V_{wp,Rd} = \frac{0.9f_{y,wc}A_{vc}}{\sqrt{3} \gamma_{M0}} \]

where

- $A_{vc}$ is the shear area of the column, see EN 1993-1-1 §6.2.6(3) and section 4.2 in this document.
7. **Rafter web in compression**

The design resistance and failure mode of the rafter web in compression, due to the reinforcement of the haunch, can be determined following the methodology given in section 5.1 for column web in transverse compression:

![Figure 7.1 Failure of the rafter web in compression](image)

\[
F_{c,wb,Rd} = \min \left( \frac{\omega k_{wc} b_{eff,c,wb} f_{y,wb}}{\gamma M_0}, \frac{\omega k_{wc} \rho b_{eff,c,wb} f_{y,wb}}{\gamma M_1} \right)
\]

If this resistance is not enough to support the acting compression force in the rafter web, a compression stiffener should be provided.

8. **Force distribution in bolt rows**

The potential resistance in each bolt row \(F_{i,Rd}\) (see section 4) is calculated one row at a time, starting at the top and working down. The force permitted in any bolt row is based on its potential resistance, and not on its lever arm. Bolts rows near a point of stiffness, such as the beam flange or a stiffener, will be therefore attract more load and have higher potential resistance.

**Plastic distribution**

A plastic distribution of forces in bolt rows is permitted, but this is only possible if the deformation of the column flange or end plate can take place.

There are two conditions that the effective tension resistance of the bolts must satisfy (see Figures 8.1(a) and 8.2(a):

According to **EN 1993-1-8 8.6.2.7.2(7)** compression resistance or the shear resistance of the column web panel must be greater than the sum of the tension resistance of all the bolts:

1. Compression:

\[
\sum F_{i,Rd(row)} \leq \min( F_{c,wc,Rd}; F_{c,fbh,Rd}; F_{c,wb,Rd} )
\]
2. Column web panel in shear:

\[ \sum F_{t,Rd(rows)} \leq \frac{V_{wp,Rd}}{\beta} \]

See sections 5 and 6 for the calculation of the compression resistance and the shear resistance of the web panel.

\[ \beta = 1 \]

is the transformation parameter according to EN 1993-1-8 §5.3(8)

If the conditions mentioned above are not satisfied then modifications are required (see Figures 8.1(b) and 8.2(b)).

**Triangular limit**

According to §6.2.7.2(9) of EN 1993-1-8, no bolt row should have a potential tension resistance greater than 1.9 times the effective tension resistance of any of the bolt rows below:

\[ F_{tx,Rd} \leq 1.9 F_{t,Rd} \]

where

\[ F_{tx,Rd} \] is the effective design tension resistance of bolt row \( x \)

\( x \) is the furthest bolt row from the centre of compression that has an effective tension resistance greater than 1.9 times the effective tension resistance of any of the bolts below.

If the potential resistance of a bolt row is governed by mode 3 failure (i.e. bolt failure) (given as \( F_{t,Rd(rows)} \geq 1.9 F_{t,Rd} \)) then plastic distribution is not possible. Therefore modification to the potential resistance is made to ensure that they do not exceed the triangular distribution for rows below the rafter flange (see Figures 8.1 and 8.2).
Because $F_{c,Rd}$ and $V_{wp,Rd} \geq F_{t,Rd,i}$ therefore the effective tension resistance ($F_{tr,Rd}$) is equal to the potential design resistance ($F_{t,Rd,i}$)

Because $F_{c,Rd}$ and/or $V_{wp,Rd} < F_{t,Rd,i}$ therefore the effective tension resistances ($F_{tr,Rd}$) have to be reduced, starting from the closest bolt to the compression centre:

- Because $F_{tx,Rd} > 1.9 F_{t,Rd}$ the effective tension resistance has to be reduced:
  \[ F_{tr,Rd} = F_{tx,Rd} \frac{h_x}{h_x} \]

- Because $F_{c,Rd}$ and/or $V_{wp,Rd} < F_{t,Rd,i}$ the effective tension resistances ($F_{tr,Rd}$) have to be reduced, starting from the closest bolt to the compression centre:
  
  **Figure 8.1**  Flush end plate – force distribution in bolt rows.
Because $F_{t,Rd}$ and $V_{wp,Rd} \geq F_{t,Rd,i}$ therefore the effective tension resistance ($F_{tr,Rd}$) is equal to the potential design resistance ($F_{t,Rd,i}$).

Because $F_{t,Rd}$ and/or $V_{wp,Rd} < F_{t,Rd,i}$ therefore the effective tension resistances ($F_{tr,Rd}$) have to be reduced starting from the closest bolt to the compression centre.

Because $F_{t,Rd} > 1.9 F_{t,Rd}$ therefore the effective tension resistance has to be reduced:

$$F_{tr,Rd} = F_{tx,Rd} \frac{h_x}{h_t}$$

Because $F_{t,Rd} > 1.9 F_{t,Rd}$ therefore the effective tension resistance has to be reduced:

$$F_{tr,Rd} = F_{tx,Rd} \frac{h_x}{h_t}$$

Figure 8.2  Extended end plate – force distribution in bolt rows.
9. Assessment of the shear resistance

The design shear resistance to vertical shear forces of the joint must be determined by accounting the contributions of the relevant basic components:

\[ V_{Rd} = n_s \min(F_{v,i,Rd}, F_{b,i,fc,Rd}, F_{b,i,cp,Rd}) \]; see Table 9.1

where

\( n_s \) is the number of bolts that are required to resist shear, see EN 1993-1-8 §6.2.2(2)

### Table 9.1 Components of the joint involved in the assessment of the shear resistance

<table>
<thead>
<tr>
<th>Component</th>
<th>Section number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolts in shear</td>
<td>( F_{v,Rd} )</td>
</tr>
<tr>
<td>Bolts in bearing on column flange</td>
<td>( F_{b,c,Rd} )</td>
</tr>
<tr>
<td>Bolts in bearing on end-plate</td>
<td>( F_{b,cp,Rd} )</td>
</tr>
</tbody>
</table>

9.1 Bolts in shear

The design resistance of bolts in shear is given in EN 1993-1-8 §3.6 as:

\[ F_{v,Rd} = \frac{\alpha_v f_{ub} A_s}{\gamma M_2} \]

where:

\( A_s \) is the tensile stress area of the bolt

\( \alpha_v = 0.6 \) for bolt classes 4.6, 5.6 and 8.8

\( \alpha_v = 0.5 \) for classes 4.8, 5.8, 6.8 and 10.9

9.2 Bolts in bearing on column flange

The design resistance of bolts in bearing on the column flange is given by the following expression according to Table 3.4 of EN 1993-1-8.

\[ F_{b,c,Rd} = \frac{k \alpha_b f_u d t_{fc}}{\gamma M_2} \]

where:

\( \alpha_b = \min(\alpha_d; \frac{f_{ub}}{f_u}; 1.0) \)

\( \alpha_d = \frac{e_1}{3d_0} \) for end bolts; \( \alpha_d = p_1 \frac{1}{4} \) for inner bolts
perpendicular to the direction of load transfer:

\[ k_1 = \min \left( \frac{2.8 e_2}{d_0} - 1.7; \ 2.5 \right) \text{ for edge bolts} \]

\[ k_1 = \min \left( \frac{1.4 p_2}{d_0} - 1.7; \ 2.5 \right) \text{ for inner bolts} \]

### 9.3 Bolts in bearing on end-plate

The design resistance for bolts subjected to shear on the end-plate, can be determined following the methodology given in the section 9.2 for bolts in bearing in the column flange:

\[ F_{b,\text{ep,Rd}} = \frac{k_1 a_b f_u d t_p}{\gamma_{M2}} \]

### 10. Limits of application

The application of this document must be in accordance with the rules and relevant limits of application set out in EN 1993-1-8. A summary of these is presented below:

- Haunches should be arranged according to EN 1993-1-8 §6.2.6.7(2):
  - the steel grade of the haunch should match that of the member;
  - the flange size and the web thickness of the haunch should not be less than that of the member;
  - the angle of the haunch flange to the flange of the member should not be greater than 45°;
  - the length of stiff bearing \( s \) should be taken as equal to the thickness of the haunch flange parallel to the beam.

- According to EN 1993-1-8 §6.2.6.7(2), the method given in this document for determining the design moment resistance of a joint \( M_{j,Rd} \) should not be used if the axial force in the connected member exceeds 5% of the design plastic resistance \( N_{pl,Rd} \) of its cross-section.

- According to EN 1993-1-8 §6.2.6.7(3) the following conservative method may be used, if the axial force \( N_{Ed} \) in the connected beam exceeds 5% of the design resistance, \( N_{pl,Rd} \):

\[ \frac{M_{j,Ed}}{M_{j,Rd}} + \frac{N_{j,Ed}}{N_{j,Rd}} \leq 1.0 \]

where:

- \( M_{j,Rd} \) is the design moment resistance of the joint, assuming no axial force;
- \( N_{j,Rd} \) is the axial design resistance of the joint, assuming no applied moment.

- Bolts in the tension zone are assumed to provide their full design resistance in tension and the total shear resistance is assumed to be provided by the bolts in the shear zone.
11. Background.

The rules in this NCCI are based on:


Annex A

Tension flange to end plate weld

1. Design a weld to carry the tension capacity of the flange

\[ N_{pl,Rd} = \frac{A_{th} f_y}{\gamma_{M0}} \]

2. Design a weld to carry the total tension force in the top three bolt rows for an extended end plate:

\[ \sum F_{tr,Rd} = F_{t1,Rd} + F_{t2,Rd} + F_{t3,Rd} \]

or the total tension force in the top two bolt rows for a flush end plate:

\[ \sum F_{tr,Rd} = F_{t1,Rd} + F_{t2,Rd} \]

According to the simplified method in §4.5.3 of EN 1993-1-8, the design resistance of the weld per unit length, \( F_{w,Rd} \) is:

\[ F_{w,Rd} = f_{vw,d} a \]

where:

\[ F_{w,Ed} \leq F_{w,Rd} \]

\( F_{w,Ed} \) is the design value of the weld force per unit length;

\( f_{vw,d} \) is the design shear resistance of the weld: \( f_{vw,d} = \frac{f_u / \sqrt{3}}{\beta_w \gamma_{M2}} \)

\( f_u \) is the nominal ultimate tensile strength of the weaker part joined

\( \beta_w \) is the correlation factor, see Table 4.1 in EN 1993-1-8.

The length of the weld to multiply with the design resistance per unit length to obtain the total design resistance of the weld is:

\[ b_{eff} = b - 2a \]

where

\( b \) is the total length of the weld

\( a \) is the throat of the weld

If the size of the weld is too big (\( a \geq 12\text{mm} \)) then the use of partial depth penetration butt welds reinforced by superimposed fillet welds is recommended. The design resistance of butt welds is given in EN 1993-1-8 §4.7.
## Quality Record

<table>
<thead>
<tr>
<th>ORIGINAL DOCUMENT</th>
<th>TRANSLATED DOCUMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>RESOURCE TITLE</strong></td>
<td>NCCI: Design of portal frame eaves connections</td>
</tr>
</tbody>
</table>

**Reference(s)**

**ORIGINAL DOCUMENT**

<table>
<thead>
<tr>
<th>Created by</th>
<th>Name</th>
<th>Company</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Jaime Grijalvo</td>
<td>LABEIN</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Jose Antonio Chica</td>
<td>LABEIN</td>
<td></td>
</tr>
</tbody>
</table>

**Technical content checked by**

<table>
<thead>
<tr>
<th>Created by</th>
<th>Name</th>
<th>Company</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Jose Antonio Chica</td>
<td>LABEIN</td>
<td></td>
</tr>
</tbody>
</table>

**Editorial content checked by**

**Technical content endorsed by the following STEEL Partners:**

1. UK
   - Name: G W Owens
   - Company: SCI
   - Date: 23/5/06

2. France
   - Name: A Bureau
   - Company: CTICM
   - Date: 23/5/06

3. Sweden
   - Name: B Uppfeldt
   - Company: SBI
   - Date: 23/5/06

4. Germany
   - Name: C Müller
   - Company: RWTH
   - Date: 23/5/06

5. Spain
   - Name: J Chica
   - Company: Labein
   - Date: 23/5/06

**Resource approved by Technical Coordinator**

<table>
<thead>
<tr>
<th>Created by</th>
<th>Name</th>
<th>Company</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>G W Owens</td>
<td>SCI</td>
<td>12/7/06</td>
</tr>
</tbody>
</table>

**TRANSLATED DOCUMENT**

This Translation made and checked by:

Translated resource approved by: