Bolted Column Splices with Minor Axis Bending

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Introduction
In multistory construction columns splices are provided for convenience of fabrication, transport and erection. If required the splices are located just above the floor level, which enables easy access to the joint.

There are two types of splice connection, full contact bearing and non-bearing. As the name suggests in bearing splices the axial load is transferred directly to the column below by full bearing contact, bolts and plates are intended to hold columns in place. In non-bearing splices the load is transferred through bolts and splice plates.

The design of bolted column splices with minor axis bending moment, ie two way moment resisting frames, is a complex process. There is not much literature available on this topic. A design guide that briefly discusses this topic is the South African Institute of Steel Construction’s Structural Steel Connections Guide or the “Green Book”. Another source of guidance is the Steel Construction Institute Publication 207, which covers the non-bearing splices with minor axis bending.

This article will cover design of bearing and non-bearing splice connections with minor axis bending based on the procedures outlined in the aforementioned guides, and relate the design equations to NZS 3404.

Bearing Splice (Full Contact)
This type of splice is the simpler, the load from upper column is transferred to the lower column through direct bearing, and the bolts and splice plates are intended to hold the two members in place. Figure 1 shows forces on the flange of column due to the minor axis bending in a full contact bearing splice. $N'_{cf} = N_{cf}/2$ and $M'_{yf} = M_{y}/2$, are the axial compressive force and bending moment per flange.

Note bolts on the bottom member are removed for clarity.

The dimension $2e$ or $b_y$ denotes the part of flange that will be in compression (Figure 1) (SAISC, 2012), with the bearing capacity determined as per NZS3404 Eq. 5.13.3.1(1), given as Eq. 1 herein:

$$\phi R_{by} = \phi1.25(2e)tf_y$$  \hspace{1cm} Eq. 1

Where:
- $t_f$ = Thickness of flange
- $f_y$ = Steel Yield Strength

From vertical force equilibrium:

$$N'_{cf} + R_t = \phi2.5et_fy$$  \hspace{1cm} Eq. 2
Where:

\( R_t = \text{Combined resultant force in the furthest line of bolts.} \)

\[ R_t = \phi 2.5t_y f_y - N_{ct} \]  \hspace{1cm} \text{Eq. 3}

Now taking moments about X:

\[ M_{mf}^* - \left( N_{ct} \left( \frac{b_t}{2} - \varepsilon \right) \right) - \left( R_t \left( \frac{b_t}{2} - \varepsilon + \frac{s_g}{2} \right) \right) = 0 \]  \hspace{1cm} \text{Eq. 4}

From substituting \( R_t \) from Eq.3 in Eq. 4, the following expression is derived for \( \varepsilon \):

\[ \varepsilon = \frac{\left( \beta - \sqrt{\beta^2 - 4\alpha\delta} \right)}{2\alpha} \]  \hspace{1cm} \text{Eq. 5}

Where:

\[ \alpha = \phi 2.5t_y f_y \]
\[ \beta = \phi 1.25t_y f_y \left( b_t + s_g \right) \]
\[ \delta = M_{mf}^* + N_{ct}^* \frac{s_g}{2} \]

Input the value of \( \varepsilon \) from Eq. 5, into Eq. 3 to determine the magnitude of the combined force, \( R_t \) in the furthest line of bolts that must be resisted by all bolts on that line.

If Eq. 3 yields a negative answer, then the minor axis bending does not govern (SAISC, 2012).
**Worked Example**

Consider a 250 UC 90 column with following design actions:

Axial Compression \[ N'_C = 750kN \]
Minor Axis Bending \[ M'_y = 100kNm \]
Axial Compression/flange \[ N'_c = 375kN \]
Minor Axis Bending/flange \[ M'_f = 50kNm \]

**Column Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flange width</td>
<td>256mm</td>
</tr>
<tr>
<td>Flange Thickness</td>
<td>17.3mm</td>
</tr>
<tr>
<td>Yield Strength</td>
<td>300MPa</td>
</tr>
<tr>
<td>Bolt Gauge</td>
<td>140mm</td>
</tr>
</tbody>
</table>

Determine the flange compression width, \( 2\varepsilon \):

\[
\alpha = \phi 2.5t f_y = 0.9 \times 2.5 \times 17.3 \times 300 = 11.68 \times 10^3 \frac{N}{mm} 
\]

\[
\beta = \phi 1.25t f_y \left( b_f + s_g \right) = 0.9 \times 1.25 \times 17.3 \times 300 \left( 256 + 140 \right) = 2312 \times 10^3 N 
\]

\[
\delta = M'_f + N'_c \frac{s_g}{2} = 50 \times 10^6 + 375000 \frac{140}{2} = 76.25 \times 10^6 Nmm 
\]

\[
\varepsilon = \left( \beta - \sqrt{\beta^2 - 4 \alpha \delta} \right) \frac{2}{2 \alpha} = \frac{2312 \times 10^3 - \sqrt{(2312 \times 10^3)^2 - 4 \times 11.7 \times 10^3 \times 76.3 \times 10^6}}{2 \times 11.7 \times 10^3} = 41.8 \text{mm} 
\]

\[
R_i = \phi 2.5(\varepsilon) t f_y - N'_c = 0.9 \times 2.5 \times 41.8 \times 17.3 \times 300 - 375000 = 113kN 
\]

Each line of bolts must resist 113kN load.
Non-Bearing Splice
In non-bearing splice connections, it is generally assumed that no contact exists between the two columns, and any bearing between them is ignored. These splices are detailed with a gap between the columns. All axial load and moment is to be transferred to the lower column through the bolts and splice plates. A typical non-bearing column splice is shown in Figure 2.

Unlike bearing splices the load is not equally shared by the all bolts and there will be some remote bolt(s) which carry more load.

The load on each bolt in this case is derived from the bolt group geometry. SCI Publication 207 provides guideline on determining the worst reaction on the remote bolt.

![Figure 2: Column Splice with Minor Axis Bending & Maximum Bolt Resultant](image)

Where:
- \( b_f \) Splice plate width
- \( d_h \) Bolt hole diameter
- \( a_{e1} \) & \( a_{e3} \) Edge distances, as defined in Figure 2

Because of the symmetry of the bolt group geometry shown in Figure 2, by inspection the bottom left bolt is loaded more than the other 5 bolts, this would reverse if the loads are reversed. If the bolt group is not symmetrical the force on each bolt should be determined to verify the worst load on bolt.

Flange Plates
The flange splice plates must satisfy the following equation:

\[
\frac{N_{cf}^*}{\phi N_{cf}} + \frac{M_{yf}^*}{\phi M_{yf}} \leq 1.0
\]

Eq. 6

Where:
- \( N_{cf}^* \) Maximum axial force/flange (Could be Tension or Compression)
- \( \phi N_{cf} \) Axial capacity of the plate (based on net area – minus bolt hole dia)
- \( M_{yf}^* \) Maximum weak axis bending/flange
- \( \phi M_{yf} \) Bending capacity of the plate (based on net Section Modulus – minus bolt hole dia)
It is not necessary to carry out this check, if the flange plates are equal, with respect to area & modulus, to the flange itself and the steel grades are the same (SCI P-207, 1997).

Bolt Hole Diameter \( d_h = \begin{cases} d + 2\text{mm} & \text{For } d \leq 24 \\ d + 3\text{mm} & \text{For } d > 24 \end{cases} \)

Where \( d = \) diameter of bolt.

**Maximum Bolt Force**

The maximum resultant force on bolt is calculated as per Eq. 7:

\[
R = \sqrt{r_y^2 + r_x^2 + \left(2r_x r_y \cos \theta \right)^2} \leq \phi \Psi_{res} \quad \text{Eq. 7}
\]

Where:

\[
\begin{align*}
   r_y &= \frac{N_{yf}}{n} \\
   r_x &= \frac{M_{xf}}{Z_b}
\end{align*}
\]

\( n = \) number of bolts

\( Z_b = \) Elastic modulus of bolt group

\( s_{pf} = \) bolt centre – centre distance

\( \theta = \) Angle from the centre of bolt group to the remote bolt (see Figure 2).

\( \phi \Psi_{res} = \) Resultant bolt shear and bearing capacity, which is the less of:

a) The slip resistance of the bolt per interface, if TF connection

b) The bearing capacity of the bolt in the flange plate(s), reduced where required by bolt hole tearing for the resultant end distance

c) The bearing capacity of the bolt in the flange, reduced where required by bolt hole tearing for the resultant end distance.

**References**

