Bolted Column Splices with Minor Axis Bending

Author: *Zahid Hamid, Kevin Cowie*
Affiliation: *Steel Construction New Zealand Inc.*
Date: *19th February 2015*
Ref: *CON3102*

Key Words
Splices, Bolted Splice, Bolted Column Splice, Minor Axis Splice

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. $N^*_{cf} = N^*_c/2$ 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 2ε or b_s, 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:

$$\phi R_{by} = \phi 1.25(2\varepsilon)t_f f_y \qquad \text{Eq. 1}$$

Where:

t_f = Thickness of flange

f_y = Steel Yield Strength

From vertical force equilibrium:

$$N^*_{cf} + R_t = \phi 2.5\varepsilon t_f f_y \qquad \text{Eq. 2}$$

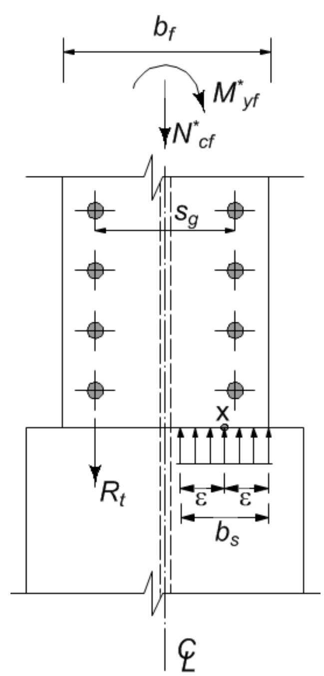

Figure 1: Forces/Flange of Column Due to Minor Axis Bending

Where:

R_t = Combined resultant force in the furthest line of bolts.

$$R_t = \phi 2.5 \varepsilon t_f f_y - N^*_{cf} \qquad \text{Eq. 3}$$

Now taking moments about X:

$$M^*_{yf} - \left(N^*_{cf}\left(\frac{b_f}{2} - \varepsilon\right)\right) - \left(R_t\left(\frac{b_f}{2} - \varepsilon + \frac{s_g}{2}\right)\right) = 0 \qquad \text{Eq. 4}$$

From substituting R_t from Eq.3 in Eq. 4, the following expression is derived for ε:

$$\varepsilon = \frac{\left(\beta - \sqrt{\beta^2 - 4\alpha\delta}\right)}{2\alpha} \qquad \text{Eq. 5}$$

Where:

$$\alpha = \phi 2.5 t_f f_y$$

$$\beta = \phi 1.25 t_f f_y \left(b_f + s_g\right)$$

$$\delta = M^*_{yf} + N^*_{cf}\frac{s_g}{2}$$

Input the value of ε 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	$N_{cf}^* = 375kN$
Minor Axis Bending/flange	$M_{yf}^* = 50kNm$

Column Properties

Flange width	$b_f = 256mm$
Flange Thickness	$t_f = 17.3mm$
Yield Strength	$f_y = 300MPa$
Bolt Gauge	$s_g = 140mm$

Determine the flange compression width, 2ε:

$$\alpha = \phi 2.5 t_f f_y = 0.9x2.5x17.3x300 = 11.68x10^3 \frac{N}{mm}$$

$$\beta = \phi 1.25 t_f f_y \left(b_f + s_g\right) = 0.9x1.25x17.3x300\left(256+140\right) = 2312x10^3 N$$

$$\delta = M_{yf}^* + N_{cf}^* \frac{s_g}{2} = 50x10^6 + 375000\frac{140}{2} = 76.25x10^6 Nmm$$

$$\varepsilon = \frac{\left(\beta - \sqrt{\beta^2 - 4\alpha\delta}\right)}{2\alpha} = \frac{\left(2312x10^3 - \sqrt{\left(2312x10^3\right)^2 - 4x11.7x10^3x76.3x10^6}\right)}{2x11.7x10^3} = 41.8mm$$

$$R_t = \phi 2.5\left(\varepsilon\right) t_f f_y - N_{cf}^* = 0.9x2.5x41.8x17.3x300 - 375000 = 113kN$$

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.

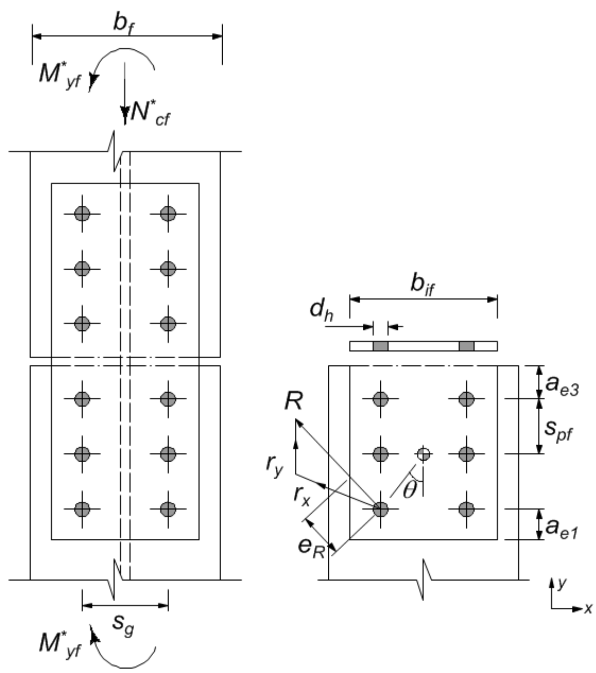

Figure 2: Column Splice with Minor Axis Bending & Maximum Bolt Resultant

Where:

b_{if}	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:

$$\frac{N^*_{cf}}{\phi N_{sf}} + \frac{M^*_{yf}}{\phi M_{yf}} \leq 1.0 \qquad \text{Eq. 6}$$

Where:

N^*_{cf} = Maximum axial force/flange (Could be Tension or Compression)

ϕN_{sf} = Axial capacity of the plate (based on net area – minus bolt hole dia)

M^*_{yf} = Maximum weak axis bending/flange

ϕM_{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 = d + 2mm For $d \le 24$
d + 3mm For $d > 24$

Where d = diameter of bolt.

Maximum Bolt Force

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

$$R = \sqrt{r_x^2 + r_y^2 + \left(2 r_x r_y \cos\theta\right)} \le \phi V_{res} \qquad \text{Eq. 7}$$

Where:

$r_y = \frac{N^*_{cf}}{n}$

$r_x = \frac{M^*_{yf}}{Z_b}$

n = number of bolts

Z_b = Elastic modulus of bolt group

s_{pf} = bolt centre – centre distance

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

ϕV_{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

SAISC, Structural Steel Connections, The South African Institute of Steel Construction, Johannesburg, South Africa, 2012.

SCI, Joints in Steel Construction Moment Connections, the Steel Construction Institute Publication 207, London, UK 1997.

SNZ, Steel Structures Standard (Incorporating Amendments 1 and 2), NZS 3404:1997, Standards New Zealand, Wellington, 2007.