S12

Beam - Column Connection - Ver W3.0.06 - 06 Dec 2016

Title : UB CONNECTIONS
Code of Practice : SABS 0162 - 1993
Created : 23/06/2022 22:55:03

Notes and Assumptions

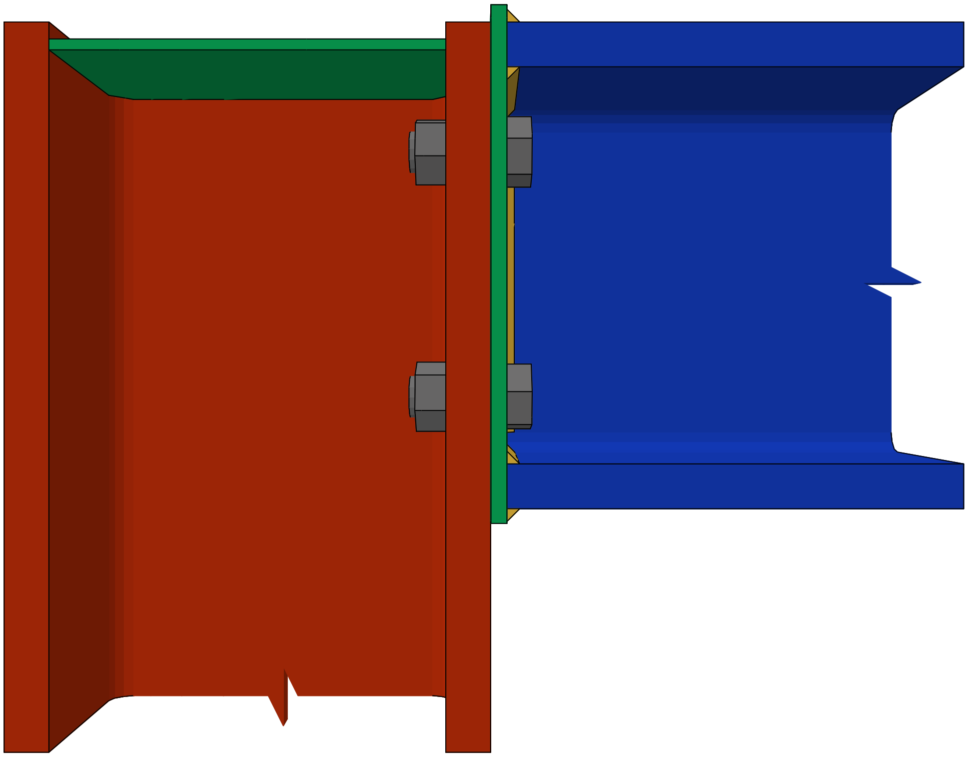
1 All references are formated "EC3 Part : Section" eg: 8 : 3.6.2(3)a. for Eurocode 1993-1-8 Section 3.6.2(3)a.
2 All bolt holes are assumed to be normal clearance holes.
3 All bolts are assumed to have threads in their shear planes.
4 It is assumed that the connection is deep enough for the flanges to resist the compressive and tensile forces in them.
5 It is assumed that compressive forces in flanges and stiffeners are conveyed through welds and not through bearing.
6 Axial force in the column is not considered in the design.

Summary

Summary of Forces and Capacities for Design to SABS 0162 - 1993

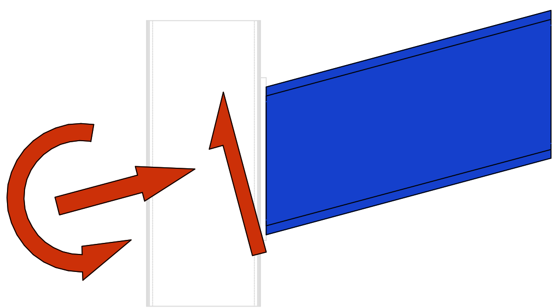
Check	Member	Type	LC	Applied	Capacity	Units	% of Cap.	?
1	Weld	Flange	1	70.9	351.2	kN	20.2	O.K.
2	Weld	Web	1	10	294.2	kN	3.4	O.K.
3	Column Web	Tension Yielding	1	67.8	856.7	kN	7.9	O.K.
4	Column Web	Compression Crippling	1	70.9	820.6	kN	8.6	O.K.
5	Column Web	Compression Buckling	1	70.9	8846.2	kN	0.8	O.K.
6	Column Web	Shear	1	70.9	600.8	kN	11.8	O.K.
7	Bolts & Flange	Tension & Bending	1	79.9	131.3	kN	60.9	O.K.
8	Column Flange	Bearing	1	2.5	395.6	kN	0.6	O.K.
9	Bolts & End Plate	Tension & Bending	1	79.9	81.6	kN	97.9	O.K.
10	End Plate	Bearing	1	2.5	138.9	kN	1.8	O.K.
11	Bolts	Shear	1	2.5	39.4	kN	6.3	O.K.
12	Bolts	Shear & Tension	1	0.7	1.4	kN	48	O.K.
13	Bolts	Slip	N/A	N/A	N/A	kN	N/A	N/A

Input



General Settings	
Bolt Tension Analysis	Plastic
Bolt Type	Bearing
Bolt Grade	4.8
Member Ultimate Strength	480
Member Yield Strength	350
Weld Ultimate Strength	480

Connection Type	Flush End Plate
Column	203x203x86 H1
Beam	203x203x86 H1
Column Extent Above (mm)	0
Beam Angle	0
Haunch Depth (mm)	0
Haunch Length (mm)	0



Ultimate Limit State Loads in Beam				SLS Factor (Divide to get Loads)
Load Case	Shear (kN)	Axial (kN)	Moment (kNm)	
1	10	3.1	14	1.5

End Plate	Width	(mm)	220.9
	Extent Above Beam Flange	(mm)	N/A
	Extent Below Beam Flange	(mm)	N/A
	Thickness	(mm)	7.2
	Stiffeners		N/A
Column Stiffeners	Width	(mm)	98
	Top Stiffener Thickness	(mm)	5
	Bottom Stiffener Thickness	(mm)	None
	Shear Stiffener Thickness	(mm)	None
	Shear Stiffener Orientation		None
Web Plates	Layout		None
	Thickness	(mm)	5
Top Backing Plate	Thickness	(mm)	None
Bottom Backing Plate	Thickness	(mm)	None
Bolts	Diameter	(mm)	20
Rows of Bolts	Above Top Flange		N/A
	Below Top Flange		1
	Above Bottom Flange		1
	Below Bottom Flange		N/A
	Row Spacing	(mm)	N/A
Bolt Offsets	Web	(mm)	50
	Flange	(mm)	30
	Above Haunch	(mm)	N/A
Welds	Beam Flanges		6
	Beam Web	(mm)	6
	Top Stiffener	(mm)	0
	Bottom Stiffener	(mm)	N/A
	Shear Stiffener	(mm)	N/A

Capacity of the Stiffeners

The capacity of the applicable stiffener is the lesser of the capacity of the plate and the welds.

Top Stiffener : Weld

The Capacity of the weld is the lesser of : *13.13.1*

Table 3 (a)

$$V_r = \frac{0.67 \cdot \phi_w \cdot A_w \cdot f_{uw}}{1000}$$

$$= \frac{0.67 \times 0.67 \times 0 \times 480}{1000}$$

$$= 0.0000 \times 10^0 \text{ kN}$$

Table 3 (b)

$$V_r = \frac{0.67 \cdot \phi \cdot A_m \cdot f_y}{1000}$$

$$= \frac{0.67 \times 0.9 \times 0 \times 350}{1000}$$

$$= 0.0000 \times 10^0 \text{ kN}$$

Top Stiffener : Tension

$$T_s = \frac{0.9 \cdot 2 \cdot t \cdot b \cdot f_y}{1000}$$

$$= \frac{0.9 \times 2 \times 5 \times 98 \times 350}{1000}$$

$$= 308.700 \text{ kN}$$

Top Stiffener : Compression

$$C_s = \frac{0.9 \cdot 2 \cdot t \cdot b \cdot f_y}{1000}$$

$$= \frac{0.9 \times 2 \times 5 \times 98 \times 350}{1000}$$

$$= 308.700 \text{ kN}$$

Check 1 : Capacity of the Beam Flange Welds

The worst load is encountered for Load Case : 1
when F_{max} =**70.926 kN**

The Capacity of the weld is the lesser of : *13.13.1*

Table 3 (a)

$$V_r = \frac{0.67 \cdot \phi_w \cdot A_w \cdot f_{uw}}{1000}$$

$$= \frac{0.67 \times 0.67 \times 1\,630.023 \times 480}{1000}$$

$$= 351.224 \text{ kN}$$

Table 3 (b)

$$V_r = \frac{0.67 \cdot \phi \cdot A_m \cdot f_y}{1000}$$

$$= \frac{0.67 \times 0.9 \times 2\,305.2 \times 350}{1000}$$

$$= 486.512 \text{ kN}$$

Beam Flange Weld is safe

Check 2 : Capacity of the Beam Web Welds

The worst load is encountered for Load Case : 1
when F_{max} =**10 kN**

The Capacity of the weld is the lesser of : *13.13.1*

Table 3 (a)

$$V_r = \frac{0.67 \cdot \phi_w \cdot A_w \cdot f_{uw}}{1000}$$

$$= \frac{0.67 \times 0.67 \times 1\,365.282 \times 480}{1000}$$

$$= 294.180 \text{ kN}$$

Table 3 (b)

$$V_r = \frac{0.67 \cdot \phi \cdot A_m \cdot f_y}{1000}$$

$$= \frac{0.67 \times 0.9 \times 1\,930.8 \times 350}{1000}$$

$$= 407.495 \text{ kN}$$

Beam Web Weld is safe

Check 3 : Capacity of the Column web in tension

Opposite Top flange of the beam :

The worst load is encountered for Load Case : 1
when T_{max} =**67.826 kN**

The Capacity of the web is :

21.3

$$T_r = \frac{0.9 \cdot t_w \cdot l_{eff} \cdot f_y}{1000}$$

$$= \frac{0.9 \times 13 \times 209.2 \times 350}{1000}$$

$$= 856.674 \text{ kN}$$

But due to the stiffeners the resistance is :

$$T_{eff} = T_r + T_s$$

$$= 856.674 + 0$$

$$= 856.674 \text{ kN}$$

No tensile forces in the beam bottom flange

Column web is safe in tension

Check 4 : Crippling Capacity of the Column web in Compression

No compressive forces in the beam top flange

Opposite Bottom flange of the beam :

The worst load is encountered for Load Case : 1
when C_{max} =**70.926 kN**

The Capacity of the web is : 21.3

$$C_r = \frac{0.9 \cdot t_w \cdot l_{eff} \cdot f_y}{1000}$$

$$= \frac{0.9 \times 13 \times 200.4 \times 350}{1000}$$

$$= 820.638 \text{ kN}$$

Column web is safe in compression for crippling

Check 5 : Buckling Capacity of the Column web in Compression

No compressive forces in the top beam flange

Opposite Bottom flange of the beam :

The worst load is encountered for Load Case : 1
when B_{max} =**70.926 kN**

The Capacity of the web is : 21.3

$$B_r = \frac{0.9 \cdot 640000 \cdot t_{wc} \cdot l_{eff}}{\left[\frac{h_{wc}}{t_{wc}}\right]^2 \cdot 1000}$$

$$= \frac{0.9 \times 640000 \times 13 \times 181.2}{\left[\frac{161}{13}\right]^2 \times 1000}$$

$$= 8\ 846.245 \text{ kN}$$

Column web is safe in compression for buckling

Check 6 : Shear Capacity of the Column Web

The worst load is encountered for Load Case : 1
when V_{max} =**70.926 kN**

13.4.1.1

$$V_r = \frac{0.9 \cdot 0.66 \cdot f_y \cdot t_w \cdot h}{1000}$$

$$= \frac{0.9 \times 0.66 \times 350 \times 13 \times 222.3}{1000}$$

$$= 600.810 \text{ kN}$$

Column web shear is safe

Check 7 : Bolt tension and Column Flange Bending

The worst load is encountered for Load Case : 1

F_{max} = **79.94 kN**

The resistance is the smaller of the 3 possible failure modes :

Mode 1 : Complete yielding of the flange

$$R_1 = \frac{4 \cdot M_{pl} + 2 \cdot M_{bp}}{m} \cdot 1000$$

$$= \frac{4 \times 6.92341 + 2 \times 0}{41.84} \times 1000$$

$$= 661.894 \text{ kN}$$

Mode 2 : Bolt Failure with yielding of the flange

$$R_2 = \frac{2 \cdot M_{pl} \cdot 1000 + n \cdot 2 \cdot B_t}{m + n}$$

$$= \frac{2 \times 6.92341 \times 1000 + 47.9 \times 2 \times 65.66}{41.84 + 47.9}$$

$$= 224.393 \text{ kN}$$

Mode 3 : Bolt Failure only

$$R_3 = 2 \cdot B_t$$

$$= 2 \times 65.66$$

$$= 131.320 \text{ kN}$$

Therefore R = R3 = 131.32

Bolt tension and Column Flange bending is safe

Check 8 : Bearing on the Column Flange

The Bearing Capacity of the flange at any Bolt is the lesser of :

The worst load is encountered for Load Case : 1
when B_{max} =**2.5 kN**

13.10.1c

$$B_r = \frac{3 \cdot \phi \cdot t \cdot d \cdot f_u}{1000}$$

$$= \frac{3 \times 0.67 \times 20.5 \times 20 \times 480}{1000}$$

$$= 395.568 \text{ kN}$$

Column flange bearing is safe

Check 9 : Bolt tension and End Plate Bending

The worst load is encountered for Load Case : 1

F_{max} = **79.94 kN**

The resistance is the smaller of the 3 possible failure modes :

Mode 1 : Complete yielding of the End Plate

$$R_1 = \frac{4 \cdot M_{pl}}{m} \cdot 1000$$

$$= \frac{4 \times 1.04612}{51.25} \times 1000$$

$$= 81.648 \text{ kN}$$

Mode 2 : Bolt Failure with yielding of the End Plate

$$R_2 = \frac{2 \cdot M_{pl} \cdot 1000 + n \cdot 2 \cdot B_t}{m + n}$$

$$= \frac{2 \times 1.04612 \times 1000 + 47.9 \times 2 \times 65.66}{51.25 + 47.9}$$

$$= 84.543 \text{ kN}$$

Mode 3 : Bolt Failure only

$$R_3 = 2 \cdot B_t$$

$$= 2 \times 65.66$$

$$= 131.320 \text{ kN}$$

Therefore R = R1 = 81.648

Bolt tension and End Plate bending is safe

Check 10 : Bearing on the End Plate

The Bearing Capacity of the Plate at any Bolt is :

The worst load is encountered for Load Case : 1
when B_{max} =**2.5 kN**

13.10.1c

$$B_r = \frac{3 \cdot \phi \cdot t \cdot d \cdot f_u}{1000}$$

$$= \frac{3 \times 0.67 \times 7.2 \times 20 \times 480}{1000}$$

$$= 138.931 \text{ kN}$$

End plate bearing is safe

Check 11 : Shear Capacity of the Bolts

The worst load is encountered for Load Case : 1
when V_{max} =**2.5 kN**

13.11.2

The resistance of any bolt is :

$$V_r = \frac{0.60 \cdot \phi_b \cdot m \cdot A_b \cdot f_u}{1000}$$

$$= \frac{0.60 \times 0.67 \times 1 \times 245 \times 400}{1000}$$

$$= 39.396 \text{ kN}$$

Bolt shear is safe

Check 12 : Shear and Tension Capacity of the Bolts

The worst load is encountered for Load Case : 1

The factor must be less than or equal to **1.4** : 13.11.4

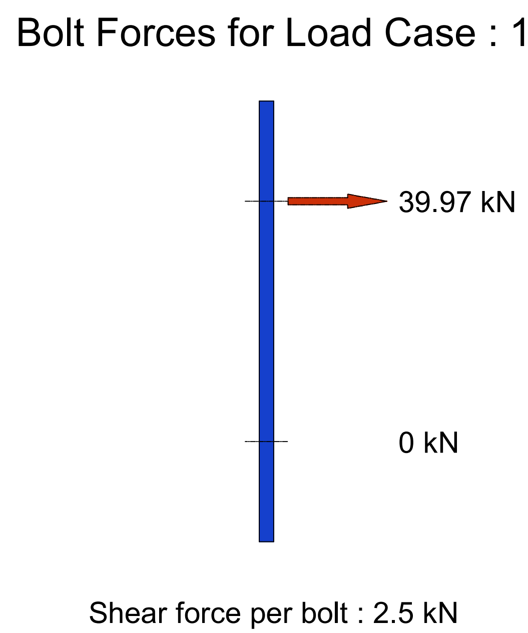
$$F_{actor} = \frac{V_u}{V_r} + \frac{T_u}{T_r}$$

$$= \frac{2.5}{39.396} + \frac{39.97}{65.66}$$

$$= 0.6722$$

Bolt shear and tension is safe

Bolt Forces


Bolt Forces for Load Case : 1
39.97 kN
0 kN
Shear force per bolt : 2.5 kN