SDS2 Steel Connection Design: Connection Cube Report

Cube: Angle web cleats
Revision: 0
Project: 0954_Validation_Eurocode_3UK
Engineer:
Fabricator: Validation_Eurocode_3UK

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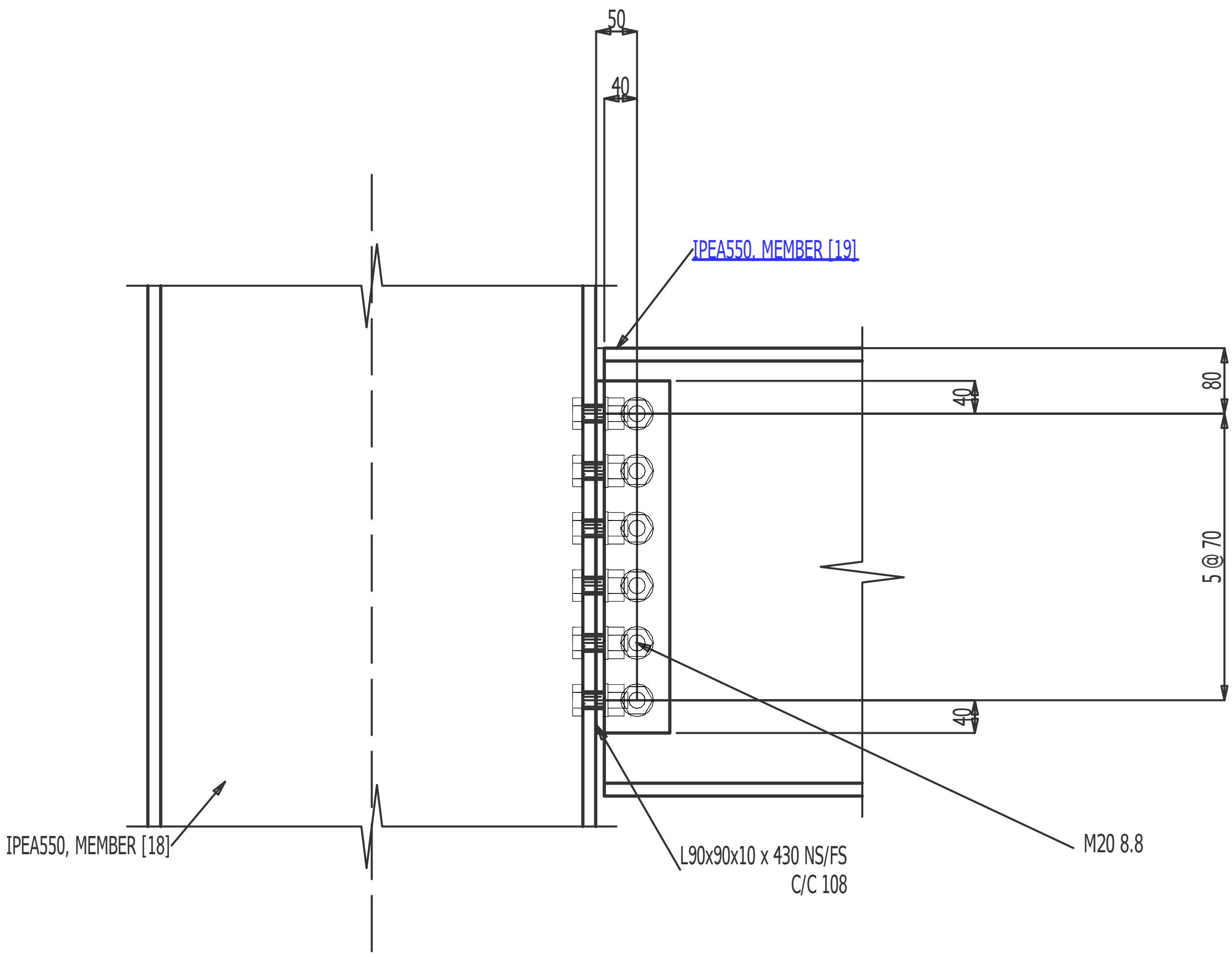
Angle web cleats [5] at X=5000, Y=10000 Elev=2227



ELEVATION VIEW

Beam B_15 [19]

Design method

- Eurocode 3 (UK NA to BS EN 1993:2005)

Overview

Section size:	IPEA550
Sequence:	1
ABM:	N/Assign
Plan length:	5000
Camber:	0.00 mm
Span length:	5000
Slope:	0.00 °
Material length:	4716
Plan rotation:	0.00 °

Section properties

Material grade:	S275
Yield stress, f_y:	275.0 MPa
Tensile strength, f_u:	430.0 MPa
Depth, h:	547.0 mm
Web thickness, t_w:	9.0 mm
Flange width, b:	210.0 mm
Flange thickness, t_f:	15.7 mm
Root radius, r:	24.0 mm
Distance between web toes of fillets, d:	467.6 mm
Moment of inertia about the major axis, I_y:	599.8 10^6 mm^4

Design summary

Left end

Connection:	Clip angle
	Bolted Web, Bolted OSL, Wide gage
	Welded extended tee: No
	Full depth extended tee: No
	Non-Safety
	Create Web Doublers: If Required
	Use web extension: If Required
	Replace doubler: No
	Skew holes in angle: Automatic
	Combine clip angles: No
	Expand Vertical bolt spacing: Auto
	Both sides, Supported, Non staggered
Elevation:	2500
Minus Dim:	274.0 mm (AUTO)
Mtrl Setback:	284.0 mm (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	450.0 kN
Moment:	0.0 kN·m (AUTO)
Tension:	0.0 kN
Compress:	0.0 kN
Tying:	370.0 kN (AUTO)

B_15 [19] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_13 [18]
Section size:	IPEA550
End 0 elevation:	0 mm
End 1 elevation:	5000 mm
Framing condition:	Flange of Column
Material grade:	S275
Supporting member thickness, t_2:	15.7 mm

Factored loads

***Shear*:**	450.0 kN
***Tying force*:**	370.0 kN

Design load notes

- Reaction has been input
- Design reaction is 41.3 % of the allowable uniform steel beam load.

Connection summary

- STANDARD CLIP ANGLE, ALL BOLTED, SHOP BOLTED TO SUPPORTED MEMBER
- TWO-SIDE CONNECTION

Connection details

Angles:	**Grade:**	S275
	Tensile strength, f_u:	430.0 MPa
	Yield stress, f_y:	275.0 MPa

Connection details (continued)

	NS angle section size:	L90x90x10 x 430
	NS outstanding leg length, $l_{osl,ns}$:	90.0 mm
	FS angle section size:	L90x90x10 x 430
	FS outstanding leg length, $l_{osl,fs}$:	90.0 mm
Bolts to supported:	**Bolt type:**	8.8, Category A
	Hole type in connection:	Standard round
	Bolt diameter, d:	20
	Bolt rows, n_1:	6
	Bolt row spacing, p_1:	70.0 mm
	Bolt columns, n_2:	1
	Web leg row edge distance, $e_{1,leg}$:	40.0 mm
Bolts to supporting:	**Bolt type:**	8.8, Category A
	Hole type in connection:	Standard round
	Bolt diameter, d:	20
	Bolt rows, n_1:	6
	Bolt row spacing, p_1:	70.0 mm
	Bolt columns, n_2:	1
	Bolt gage, p_3:	108.0 mm
	OSL row edge distance, $e_{1,osl}$:	40.0 mm
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit:	16
(at dd) Locked Via Design:	1
(at dd) Not Locked:	697

Expanded design calculation

Shear yielding of beam web (2). Reference 6.2.6, Part 1-1

Web yield stress, $f_y = 275.0\ MPa$
Bottom flange thickness, $t_{f,b} = 15.7\ mm$
Top flange thickness, $t_{f,t} = 15.7\ mm$
Web thickness, $t_w = 9.0\ mm$
Full section depth, $h = 547.0\ mm$
Applied member shear, $V_{Ed} = 450.0\ kN$
$\gamma_{M0} = 1$
Cross-sectional shear area, $A_v = 11728.8\ mm^2$
Flange width, $b = 210.0\ mm$
Flange thickness, $t_f = 15.7\ mm$
Root radius, $r_b = 24.0\ mm$
$\eta = 1$
Web depth, $h_w = h - t_{f,t} - t_{f,b}$
$= 547.0 - 15.7 - 15.7$
$= 515.6\ mm$
Web shear area, $A_{v,w} = max\left(A_v - 2 \cdot b \cdot t_f + \left(t_w + 2 \cdot r_b\right) \cdot t_f, \eta \cdot h_w \cdot t_w\right)$
$= max\left(11728.8 - 2 \cdot 210.0 \cdot 15.7 + \left(9.0 + 2 \cdot 24.0\right) \cdot 15.7, 1 \cdot 515.6 \cdot 9.0\right)$
$= 6029.7\ mm^2$

$$Unity = \left(\frac{V_{Ed}}{\left(\frac{f_y \cdot A_{v,w}}{\sqrt{3} \cdot \gamma_{M0}}\right)}\right) \cdot 1000.0$$

$$= \left(\frac{450.0}{\left(\frac{275.0 \cdot 6029.7}{\sqrt{3} \cdot 1}\right)}\right) \cdot 1000.0$$

$= 0.470048$

$$Shear\ capacity = \frac{\left(\frac{f_y \cdot A_{v,w}}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$$

$$= \frac{\left(\frac{275.0 \cdot 6029.7}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$= 957.3\ kN$

$957.3\ kN \geq 450.0\ kN$ **(OK)**
$0.470 \leq 1$ **(OK)**

Supported beam-net web shear (4). Reference 6.2.6, Part 1-1

Tensile strength, $f_u = 430.0\ MPa$

Bolt rows, $n_1 = 6$

Flange thickness, $t_f = 15.7\ mm$

Flange width, $b = 210.0\ mm$

Web thickness, $t_w = 9.0\ mm$

Full section depth, $h = 547.0\ mm$

Cross-sectional shear area, $A_v = 11728.8\ mm^2$

Flange width, $b = 210.0\ mm$

Flange thickness, $t_f = 15.7\ mm$

Root radius, $r_b = 24.0\ mm$

$\eta = 1$

Web depth, $h_w = h - t_f - t_f$

$= 547.0 - 15.7 - 15.7$

$= 515.6\ mm$

Gross shear area, $A_{v,w} = max\left(A_v - 2 \cdot b \cdot t_f + \left(t_w + 2 \cdot r_b\right) \cdot t_f, \eta \cdot h_w \cdot t_w\right)$

$= max\left(11728.8 - 2 \cdot 210.0 \cdot 15.7 + \left(9.0 + 2 \cdot 24.0\right) \cdot 15.7, 1 \cdot 515.6 \cdot 9.0\right)$

$= 6029.7\ mm^2$

Hole diameter, $d_0 = 22.0\ mm$

Net shear area, $A_{v,net} = A_{v,w} - n_1 \cdot d_0 \cdot t_w$

$= 6029.7 - 6 \cdot 22.0 \cdot 9.0$

$= 4841.7\ mm^2$

$\gamma_{M2,1\text{-}1} = 1.1$

$$Shear\ capacity = \frac{\left(\frac{f_u \cdot A_{v,net}}{\sqrt{3} \cdot \gamma_{M2,1\text{-}1}}\right)}{1000.0}$$

$$= \frac{\left(\frac{430.0 \cdot 4841.7}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$$

$= 1092.7\ kN$

Applied member shear, $V_{Ed} = 450.0\ kN$

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$

$$= \frac{450.0}{1092.7}$$

$= 0.411824$

$1092.7\ kN \geq 450.0\ kN$ **(OK)**

$0.412 \leq 1$ **(OK)**

Web leg-bearing on web (20). Reference Table 3.4, Part 1-8

Column edge distance, $e_2 = 40.0\ mm$

Connection thickness, $t = 9.0\ mm$

Connection tensile strength, $f_u = 430.0\ MPa$

Bolt row spacing, $p_1 = 70\ mm$

Bolt columns, $n_2 = 1$

Bolt rows, $n_1 = 6$

Distance from support to center of bolt group, $z = 50.0\ mm$

Bolt diameter, $d = 20.0\ mm$

Calculate bearing resistance in vertical direction

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt row spacing, $p_1 = 70.0\ mm$

$$\alpha_d = \frac{p_1}{3 \cdot d_0} - 0.25 = \frac{70.0}{3 \cdot 22.0} - 0.25 = 0.810606$$

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right) = min\left(0.810606, \frac{800.0}{430.0}, 1\right) = 0.810606$$

Bolt column spacing, $p_2 = 0.0\ mm$

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right) = min\left(\frac{2.8 \cdot 40.0}{22.0} - 1.7, 2.5\right) = 2.5$$

$\gamma_{M2,1\text{-}8} = 1.25$

Bearing resistance of inner bolt, $$F_{b,Rd,i} = \frac{\left(\frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$$

$$= \frac{\left(\frac{2.5 \cdot 0.810606 \cdot 430.0 \cdot 20.0 \cdot 9.0}{1.25}\right)}{1000.0}$$

$$= 125.5\ kN$$

End bolt resistance

$\alpha_b = min\left(\frac{f_{ub}}{f_u}, 1\right)$

$= min\left(\frac{800.0}{430.0}, 1\right)$

$= 1$

Bolt column spacing, $p_2 = 0.0\ mm$

$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right)$

$= min\left(\frac{2.8 \cdot 40.0}{22.0} - 1.7, 2.5\right)$

$= 2.5$

$\gamma_{M2,1\text{-}8} = 1.25$

Bearing resistance of end bolt, $F_{b,Rd,e} = \frac{\left(\frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$= \frac{\left(\frac{2.5 \cdot 1 \cdot 430.0 \cdot 20.0 \cdot 9.0}{1.25}\right)}{1000.0}$

$= 154.8\ kN$

Controlling bearing resistance in vertical direction, $F_{b,ver,Rd} = min\left(F_{b,Rd,e}, F_{b,Rd,i}\right)$

$= min\left(154.8, 125.5\right)$

$= 125.5\ kN$

Calculate bearing resistance in horizontal direction

Controlling row edge distance in the vertical direction, $e_1 = 0.0\ mm$

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

No inner bolts exist in this load direction.

End bolt resistance

$\alpha_d = \frac{e_2}{3 \cdot d_0}$

$= \frac{40.0}{3 \cdot 22.0}$

$= 0.606061$

$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$

$= min\left(0.606061, \frac{800.0}{430.0}, 1\right)$

End bolt resistance (continued)

$= 0.606061$

Bolt row spacing, $p_1 = 70.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

$$k_1 = min\left(\frac{1.4 \cdot p_1}{d_0} - 1.7, 2.5\right)$$

$$= min\left(\frac{1.4 \cdot 70.0}{22.0} - 1.7, 2.5\right)$$

$= 2.5$

$\gamma_{M2,1\text{-}8} = 1.25$

$$\text{Bearing resistance of end bolt, } F_{b,Rd,e} = \frac{\left(\frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$$

$$= \frac{\left(\frac{2.5 \cdot 0.606061 \cdot 430.0 \cdot 20.0 \cdot 9.0}{1.25}\right)}{1000.0}$$

$= 93.8\ kN$

Controlling bearing resistance in horizontal direction, $F_{b,hor,Rd} = F_{b,Rd,e}$

$= 93.8\ kN$

Total number of bolts, $n = n_1 \cdot n_2$

$= 6 \cdot 1$

$= 6$

$\alpha = 0$

Bolt row spacing, $p_1 = 70.0\ mm$

$$\beta = \frac{6 \cdot z}{n_1 \cdot (n_1 + 1) \cdot p_1}$$

$$= \frac{6 \cdot 50.0}{6 \cdot (6 + 1) \cdot 70.0}$$

$= 0.102041$

$$Shear\ capacity = \frac{n}{\sqrt{\left(\frac{(1 + \alpha \cdot n)}{F_{b,ver,Rd}}\right)^2 + \left(\frac{\beta \cdot n}{F_{b,hor,Rd}}\right)^2}}$$

$$= \frac{6}{\sqrt{\left(\frac{(1 + 0 \cdot 6)}{125.5}\right)^2 + \left(\frac{0.102041 \cdot 6}{93.8}\right)^2}}$$

$= 582.5\ kN$

Applied member shear, $V_{Ed} = 450.0\ kN$

Web leg-bearing on web (20). Reference Table 3.4, Part 1-8 (continued)

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$
$$= \frac{450.0}{582.5}$$
$$= 0.772533$$

$582.5\ kN \geq 450.0\ kN$ **(OK)**

$0.773 \leq 1$ **(OK)**

Connection-block tearing of web leg(s) (6). Reference 3.10.2, Part 1-8

Plate thickness, $t_p = 10.0\ mm$

Yield stress, $f_y = 275.0\ MPa$

Tensile strength, $f_u = 430.0\ MPa$

Bolt column spacing, $p_2 = 0.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt rows, $n_1 = 6$

Column edge distance, $e_2 = 40.0\ mm$

Row edge distance, $e_1 = 40.0\ mm$

Bolt columns, $n_2 = 1$

Hole diameter, $d_0 = 22.0\ mm$

Hole length, $l_h = 22.0\ mm$

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Gross shear area, $A_{gv} = 2 \cdot t_p \cdot (p_{1,total} + e_1)$
$$= 2 \cdot 10.0 \cdot (350.0 + 40.0)$$
$$= 7800.0\ mm^2$$

Net shear area, $A_{nv} = 2 \cdot (t_p \cdot (p_{1,total} + e_1) - t_p \cdot (n_1 - 0.5) \cdot d_0)$
$$= 2 \cdot (10.0 \cdot (350.0 + 40.0) - 10.0 \cdot (6 - 0.5) \cdot 22.0)$$
$$= 5380.0\ mm^2$$

Gross tensile area, $A_{gt} = 2 \cdot t_p \cdot (p_2 \cdot (n_2 - 1) + e_2)$
$$= 2 \cdot 10.0 \cdot (0.0 \cdot (1 - 1) + 40.0)$$
$$= 800.0\ mm^2$$

Net tensile area, $A_{net} = 2 \cdot (t_p \cdot (p_2 \cdot (n_2 - 1) + e_2) - t_p \cdot (n_2 - 0.5) \cdot l_h)$
$$= 2 \cdot (10.0 \cdot (0.0 \cdot (1 - 1) + 40.0) - 10.0 \cdot (1 - 0.5) \cdot 22.0)$$
$$= 580.0\ mm^2$$

$\gamma_{M2,1\text{-}1} = 1.1$

$\gamma_{M0} = 1$

$$Shear\ capacity = \frac{\left(0.5 \cdot \left(\frac{f_u \cdot A_{net}}{\gamma_{M2,1\text{-}1}}\right) + \frac{f_y \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$$

Connection-block tearing of web leg(s) (6). Reference 3.10.2, Part 1-8 (continued)

$$= \frac{\left(0.5 \cdot \left(\frac{430.0 \cdot 580.0}{1.1}\right) + \frac{275.0 \cdot 5380.0}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$$= 967.6\ kN$$

Applied member shear, $V_{Ed} = 450.0\ kN$

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$

$$= \frac{450.0}{967.6}$$

$$= 0.465069$$

$967.6\ kN \geq 450.0\ kN$ **(OK)**

$0.465 \leq 1$ **(OK)**

Shear yielding of angle(s) (15). Reference 6.2.6, Part 1-1

Connection yield stress, $f_y = 275.0\ MPa$

Connection thickness, $t = 10.0\ mm$

Connection depth, $h = 430.0\ mm$

Gross shear area, $A_{gv} = 2 \cdot t \cdot h$

$$= 2 \cdot 10.0 \cdot 430.0$$

$$= 8600.0\ mm^2$$

$\gamma_{M0} = 1$

$$Shear\ capacity = \frac{\left(\frac{\left(\frac{f_y \cdot A_{gv}}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1.27}\right)}{1000.0}$$

$$= \frac{\left(\frac{\left(\frac{275.0 \cdot 8600.0}{\sqrt{3} \cdot 1}\right)}{1.27}\right)}{1000.0}$$

$$= 1075.1\ kN$$

Applied member shear, $V_{Ed} = 450.0\ kN$

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$

Shear yielding of angle(s) (15). Reference 6.2.6, Part 1-1 (continued)

$= \frac{450.0}{1075.1}$
$= 0.418566$

$1075.1\ kN \geq 450.0\ kN$ **(OK)**

$0.419 \leq 1$ **(OK)**

Shear rupture of angle(s) (21). Reference 6.2, Part 1-1

Plate tensile strength, $f_{u,p} = 430.0\ MPa$

FS osl bolt rows, $n_{1,FS,osl} = 6$

NS osl bolt rows, $n_{1,NS,osl} = 6$

FS leg bolt rows, $n_{1,FS,leg} = 6$

NS leg bolt rows, $n_{1,NS,leg} = 6$

FS connection thickness, $t_{fs} = 10.0\ mm$

NS connection thickness, $t_{ns} = 10.0\ mm$

FS connection depth, $h_{fs} = 430.0\ mm$

NS connection depth, $h_{ns} = 430.0\ mm$

Capacity of web leg

Hole diameter, $d_0 = 22.0\ mm$

NS Net shear area, $A_{nv,ns} = t_{ns} \cdot \left(h_{ns} - n_{1,NS,leg} \cdot d_0\right)$
$= 10.0 \cdot (430.0 - 6 \cdot 22.0)$
$= 2980.0\ mm^2$

FS Net shear area, $A_{nv,fs} = t_{fs} \cdot \left(h_{fs} - n_{1,FS,leg} \cdot d_0\right)$
$= 10.0 \cdot (430.0 - 6 \cdot 22.0)$
$= 2980.0\ mm^2$

Total net shear area, $A_{nv,total} = A_{nv,ns} + A_{nv,fs}$
$= 2980.0 + 2980.0$
$= 5960.0\ mm^2$

$\gamma_{M2,1\text{-}1} = 1.1$

Web leg net shear capacity, $R_{v,web} = \frac{\left(\frac{f_{u,p} \cdot A_{nv,total}}{\sqrt{3} \cdot \gamma_{M2,1\text{-}1}}\right)}{1000.0}$

$= \frac{\left(\frac{430.0 \cdot 5960.0}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$

$= 1345.1\ kN$

Capacity of outstanding leg

Hole diameter, $d_0 = 22.0\ mm$

NS Net shear area, $A_{nv,ns} = t_{ns} \cdot (h_{ns} - n_{1,NS,osl} \cdot d_0)$

$= 10.0 \cdot (430.0 - 6 \cdot 22.0)$

$= 2980.0\ mm^2$

FS Net shear area, $A_{nv,fs} = t_{fs} \cdot (h_{fs} - n_{1,FS,osl} \cdot d_0)$

$= 10.0 \cdot (430.0 - 6 \cdot 22.0)$

$= 2980.0\ mm^2$

Total net shear area, $A_{nv,total} = A_{nv,ns} + A_{nv,fs}$

$= 2980.0 + 2980.0$

$= 5960.0\ mm^2$

$\gamma_{M2,1\text{-}1} = 1.1$

$$\text{OSL net shear capacity, } R_{v,osl} = \frac{\left(\dfrac{f_{u,p} \cdot A_{nv,total}}{\sqrt{3} \cdot \gamma_{M2,1\text{-}1}}\right)}{1000.0}$$

$$= \frac{\left(\dfrac{430.0 \cdot 5960.0}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$$

$= 1345.1\ kN$

$Shear\ capacity = min\ (R_{v,web}, R_{v,osl})$

$= min\ (1345.1, 1345.1)$

$= 1345.1\ kN$

Applied member shear, $V_{Ed} = 450.0\ kN$

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$

$$= \frac{450.0}{1345.1}$$

$= 0.334548$

$1345.1\ kN \geq 450.0\ kN$ **(OK)**

$0.335 \leq 1$ **(OK)**

Web leg-bolt shear (3). Reference Table 3.4, Part 1-8

Number of shear planes, $N_s = 2$

Bolt row spacing, $p_1 = 70\ mm$

Bolt columns, $n_2 = 1$

Bolt rows, $n_1 = 6$

Distance from weld to center of bolt group, $z = 50.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

$$\beta = \frac{6 \cdot z}{n_1 \cdot (n_1 + 1) \cdot p_1}$$

Web leg-bolt shear (3). Reference Table 3.4, Part 1-8 (continued)

$$= \frac{6 \cdot 50.0}{6 \cdot (6 + 1) \cdot 70.0}$$
$$= 0.102041$$

Tensile stress area of the bolt, $A = 244.8\ mm^2$

$\gamma_{M2,1\text{-}8} = 1.25$

$(t_{pa} = 0.0\ mm) \leq \left(\frac{d}{3} = \frac{20.0}{3} = 6.7\ mm\right)$

Not necessary to reduce shear capacity for packing.

Bolt shear resistance, $F_{v,Rd} = \frac{\left(\frac{480.0 \cdot A}{\gamma_{M2,1\text{-}8}}\right) \cdot N_s}{1000.0}$

$$= \frac{\left(\frac{480.0 \cdot 244.8}{1.25}\right) \cdot 2}{1000.0}$$

$$= 188.0\ kN$$

$$Shear\ capacity = \left(\frac{n_1 \cdot n_2}{\sqrt{1^2 + (\beta \cdot n_1 \cdot n_2)^2}}\right) \cdot F_{v,Rd}$$
$$= \left(\frac{6 \cdot 1}{\sqrt{1^2 + (0.102041 \cdot 6 \cdot 1)^2}}\right) \cdot 188.0$$
$$= 962.0\ kN$$

Applied member shear, $V_{Ed} = 450.0\ kN$

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$
$$= \frac{450.0}{962.0}$$
$$= 0.467776$$

$962.0\ kN \geq 450.0\ kN$ **(OK)**

$0.468 \leq 1$ **(OK)**

Bolts-bearing on web cover plates (20). Reference 3.6, Part 1-8

NS column edge distance, $e_{2,NS} = 40.0\ mm$

NS bottom row edge distance, $e_{1,NS,b} = 40.0\ mm$

NS top row edge distance, $e_{1,NS,t} = 40.0\ mm$

NS connection thickness, $t_{ns} = 10.0\ mm$

Connection tensile strength, $f_u = 430.0\ MPa$

Bolt row spacing, $p_1 = 70\ mm$

Bolt columns, $n_2 = 1$

NS bolt rows, $n_{1,NS} = 6$

Eccentricity in z-direction, $z = 50.0\ mm$

Bolt diameter, $d = 20.0\ mm$

Calculate bearing resistance in vertical direction

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt row spacing, $p_1 = 70.0\ mm$

$$\alpha_d = \frac{p_1}{3 \cdot d_0} - 0.25$$
$$= \frac{70.0}{3 \cdot 22.0} - 0.25$$
$$= 0.810606$$

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$$
$$= min\left(0.810606, \frac{800.0}{430.0}, 1\right)$$
$$= 0.810606$$

Bolt column spacing, $p_2 = 0.0\ mm$

$$k_1 = min\left(\frac{2.8 \cdot e_{2,NS}}{d_0} - 1.7, 2.5\right)$$
$$= min\left(\frac{2.8 \cdot 40.0}{22.0} - 1.7, 2.5\right)$$
$$= 2.5$$

$\gamma_{M2,1\text{-}8} = 1.25$

Bearing resistance of inner bolt, $F_{b,Rd,i} = \dfrac{\left(\dfrac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t_{ns}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$$= \frac{\left(\frac{2.5 \cdot 0.810606 \cdot 430.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$$

$$= 139.4\ kN$$

End bolt resistance

$$\alpha_d = \frac{e_{1,NS,t}}{3 \cdot d_0}$$
$$= \frac{40.0}{3 \cdot 22.0}$$
$$= 0.606061$$

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$$
$$= min\left(0.606061, \frac{800.0}{430.0}, 1\right)$$
$$= 0.606061$$

Bolt column spacing, $p_2 = 0.0\ mm$

End bolt resistance (continued)

$k_1 = min\left(\frac{2.8 \cdot e_{2,NS}}{d_0} - 1.7, 2.5\right)$

$= min\left(\frac{2.8 \cdot 40.0}{22.0} - 1.7, 2.5\right)$

$= 2.5$

$\gamma_{M2,1\text{-}8} = 1.25$

Bearing resistance of end bolt, $F_{b,Rd,e} = \frac{\left(\frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t_{ns}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$= \frac{\left(\frac{2.5 \cdot 0.606061 \cdot 430.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$

$= 104.2\ kN$

Controlling bearing resistance in vertical direction, $F_{b,ver,Rd} = min\left(F_{b,Rd,e}, F_{b,Rd,i}\right)$

$= min\left(104.2, 139.4\right)$

$= 104.2\ kN$

Calculate bearing resistance in horizontal direction

Controlling row edge distance in the vertical direction, $e_1 = min\left(e_{1,NS,t}, e_{1,NS,b}\right)$

$= min\left(40.0, 40.0\right)$

$= 40.0\ mm$

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

No inner bolts exist in this load direction.

End bolt resistance

$\alpha_d = \frac{e_{2,NS}}{3 \cdot d_0}$

$= \frac{40.0}{3 \cdot 22.0}$

$= 0.606061$

$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$

$= min\left(0.606061, \frac{800.0}{430.0}, 1\right)$

$= 0.606061$

Bolt row spacing, $p_1 = 70.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

End bolt resistance (continued)

$$k_1 = min\left(\frac{2.8 \cdot e_1}{d_0} - 1.7, \frac{1.4 \cdot p_1}{d_0} - 1.7, 2.5\right)$$
$$= min\left(\frac{2.8 \cdot 40.0}{22.0} - 1.7, \frac{1.4 \cdot 70.0}{22.0} - 1.7, 2.5\right)$$
$$= 2.5$$
$$\gamma_{M2,1\text{-}8} = 1.25$$

Bearing resistance of end bolt, $F_{b,Rd,e} = \frac{\left(\frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t_{ns}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$$= \frac{\left(\frac{2.5 \cdot 0.606061 \cdot 430.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$$
$$= 104.2\ kN$$

Controlling bearing resistance in horizontal direction, $F_{b,hor,Rd} = F_{b,Rd,e}$
$$= 104.2\ kN$$

Total number of bolts, $n = n_{1,NS} \cdot n_2$
$$= 6 \cdot 1$$
$$= 6$$

$\alpha = 0$

Bolt row spacing, $p_1 = 70.0\ mm$

$$\beta = \frac{6 \cdot z}{n_{1,NS} \cdot (n_{1,NS} + 1) \cdot p_1}$$
$$= \frac{6 \cdot 50.0}{6 \cdot (6 + 1) \cdot 70.0}$$
$$= 0.102041$$

$$Shear\ capacity = 2 \cdot \left(\frac{n}{\sqrt{\left(\frac{(1 + \alpha \cdot n)}{F_{b,ver,Rd}}\right)^2 + \left(\frac{\beta \cdot n}{F_{b,hor,Rd}}\right)^2}}\right)$$
$$= 2 \cdot \left(\frac{6}{\sqrt{\left(\frac{(1 + 0 \cdot 6)}{104.2}\right)^2 + \left(\frac{0.102041 \cdot 6}{104.2}\right)^2}}\right)$$
$$= 1066.8\ kN$$

Applied member shear, $V_{Ed} = 450.0\ kN$

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$
$$= \frac{450.0}{1066.8}$$

Bolts-bearing on web cover plates (20). Reference 3.6, Part 1-8 (continued)

$= 0.421823$

$1066.8\ kN \geq 450.0\ kN$ **(OK)**

$0.422 \leq 1$ **(OK)**

Outstanding leg-bolt shear (3). Reference 3.6, Part 1-8

Number of shear planes, $N_s = 1$

FS bolt columns, $n_{2,fs} = 1$

NS bolt columns, $n_{2,ns} = 1$

FS bolt rows, $n_{1,FS} = 6$

NS bolt rows, $n_{1,NS} = 6$

Tensile stress area of the bolt, $A = 244.8\ mm^2$

$\gamma_{M2,1\text{-}8} = 1.25$

$\left(t_{pa} = 0.0\ mm\right) \leq \left(\frac{d}{3} = \frac{20.0}{3} = 6.7\ mm\right)$

Not necessary to reduce shear capacity for packing.

Bolt shear resistance, $F_{v,Rd} = \dfrac{\left(\dfrac{480.0 \cdot A}{\gamma_{M2,1\text{-}8}}\right) \cdot N_s}{1000.0}$

$$= \frac{\left(\dfrac{480.0 \cdot 244.8}{1.25}\right) \cdot 1}{1000.0}$$

$= 94.0\ kN$

Allowable shear load, $P = F_{v,Rd} \cdot \left(n_{1,NS} \cdot n_{2,ns} + n_{1,FS} \cdot n_{2,fs}\right)$

$= 94.0 \cdot (6 \cdot 1 + 6 \cdot 1)$

$= 1128.0\ kN$

$Shear\ capacity = 0.8 \cdot P$

$= 0.8 \cdot 1128.0$

$= 902.4\ kN$

Applied member shear, $V_{Ed} = 450.0\ kN$

$Unity = \dfrac{V_{Ed}}{Shear\ capacity}$

$= \dfrac{450.0}{902.4}$

$= 0.498671$

$902.4\ kN \geq 450.0\ kN$ **(OK)**

$0.499 \leq 1$ **(OK)**

Outstanding leg-bearing on angles (110). Reference 3.6, Part 1-8

Tensile strength, $f_u = 430.0\ MPa$

Connection thickness, $t = 10.0\ mm$

Column edge distance, $e_2 = 41.0\ mm$

Outstanding leg-bearing on angles (110). Reference 3.6, Part 1-8 (continued)

Bolt row spacing, $p_1 = 70\ mm$

Row edge distance, $e_1 = 40.0\ mm$

Bolt diameter, $d = 20.0\ mm$

Number of sides, $N = 2$

Bolt columns, $n_2 = 1$

Bolt rows, $n_1 = 6$

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Length of joint, $l_j = p_{1,total}$

$= 350.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt row spacing, $p_1 = 70.0\ mm$

$$\alpha_d = \frac{p_1}{3 \cdot d_0} - 0.25$$
$$= \frac{70.0}{3 \cdot 22.0} - 0.25$$
$$= 0.810606$$

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$$
$$= min\left(0.810606, \frac{800.0}{430.0}, 1\right)$$
$$= 0.810606$$

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right)$$
$$= min\left(\frac{2.8 \cdot 41.0}{22.0} - 1.7, 2.5\right)$$
$$= 2.5$$

$$\gamma_{M2,1\text{-}8} = 1.25$$

Bearing resistance of inner bolt, $F_{b,Rd,i} = \dfrac{\left(\dfrac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$$= \frac{\left(\frac{2.5 \cdot 0.810606 \cdot 430.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$$

$$= 139.4\ kN$$

End bolt resistance

$$\alpha_d = \frac{e_1}{3 \cdot d_0}$$
$$= \frac{40.0}{3 \cdot 22.0}$$

End bolt resistance (continued)

$$= 0.606061$$

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$$

$$= min\left(0.606061, \frac{800.0}{430.0}, 1\right)$$

$$= 0.606061$$

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right)$$

$$= min\left(\frac{2.8 \cdot 41.0}{22.0} - 1.7, 2.5\right)$$

$$= 2.5$$

$$\gamma_{M2,1\text{-}8} = 1.25$$

$$\text{Bearing resistance of end bolt, } F_{b,Rd,e} = \frac{\left(\frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$$

$$= \frac{\left(\frac{2.5 \cdot 0.606061 \cdot 430.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$$

$$= 104.2\ kN$$

Bolt shear resistance

Bolt shear resistance, $F_{v,Rd} = 94.0\ kN$

Reduction factor, $\beta_{LF} = 0.9875$

$$\left(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 0.9875 = 74.3\ kN\right) < \left(F_{b,Rd,e} = 104.2\ kN\right)$$

$$\left(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 0.9875 = 74.3\ kN\right) < \left(F_{b,Rd,i} = 139.4\ kN\right)$$

Fastener resistance limited by bolt shear resistance

$$\text{Bearing resistance, } F_{b,Rd} = min\left(F_{b,Rd,e}, F_{b,Rd,i}, 0.8 \cdot F_{v,Rd} \cdot \beta_{LF}\right)$$

$$= min\left(104.2, 139.4, 0.8 \cdot 94.0 \cdot 0.9875\right)$$

$$= 74.3\ kN$$

$$Shear\ capacity = F_{b,Rd} \cdot n_1 \cdot n_2 \cdot N$$

$$= 74.3 \cdot 6 \cdot 1 \cdot 2$$

$$= 891.1\ kN$$

Applied member shear, $V_{Ed} = 450.0\ kN$

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$

$$= \frac{450.0}{891.1}$$

$$= 0.504994$$

$891.1\ kN \geq 450.0\ kN$ **(OK)**

$0.505 \leq 1$ **(OK)**

Outstanding leg-bearing on support (110). Reference 3.6, Part 1-8

Tensile strength, $f_u = 430.0\ MPa$

Connection thickness, $t = 15.7\ mm$

Column edge distance, $e_2 = 51.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt diameter, $d = 20.0\ mm$

Number of sides, $N = 2$

Bolt columns, $n_2 = 1$

Bolt rows, $n_1 = 6$

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Length of joint, $l_j = p_{1,total}$

$= 350.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt row spacing, $p_1 = 70.0\ mm$

$$\alpha_d = \frac{p_1}{3 \cdot d_0} - 0.25$$

$$= \frac{70.0}{3 \cdot 22.0} - 0.25$$

$$= 0.810606$$

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$$

$$= min\left(0.810606, \frac{800.0}{430.0}, 1\right)$$

$$= 0.810606$$

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right)$$

$$= min\left(\frac{2.8 \cdot 51.0}{22.0} - 1.7, 2.5\right)$$

$$= 2.5$$

$$\gamma_{M2,1\text{-}8} = 1.25$$

Bearing resistance of inner bolt, $F_{b,Rd,i} = \dfrac{\left(\dfrac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$$= \frac{\left(\dfrac{2.5 \cdot 0.810606 \cdot 430.0 \cdot 20.0 \cdot 15.7}{1.25}\right)}{1000.0}$$

$$= 218.9\ kN$$

End bolt resistance

$\alpha_b = min\left(\frac{f_{ub}}{f_u},1\right)$
$= min\left(\frac{800.0}{430.0},1\right)$
$= 1$

$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7,2.5\right)$
$= min\left(\frac{2.8 \cdot 51.0}{22.0} - 1.7,2.5\right)$
$= 2.5$

$\gamma_{M2,1\text{-}8} = 1.25$

Bearing resistance of end bolt, $F_{b,Rd,e} = \frac{\left(\frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$= \frac{\left(\frac{2.5 \cdot 1 \cdot 430.0 \cdot 20.0 \cdot 15.7}{1.25}\right)}{1000.0}$

$= 270.0\ kN$

Bolt shear resistance

Bolt shear resistance, $F_{v,Rd} = 94.0\ kN$
Reduction factor, $\beta_{LF} = 0.9875$

$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 0.9875 = 74.3\ kN) < (F_{b,Rd,e} = 270.0\ kN)$
$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 0.9875 = 74.3\ kN) < (F_{b,Rd,i} = 218.9\ kN)$

Fastener resistance limited by bolt shear resistance

Bearing resistance, $F_{b,Rd} = min\ (F_{b,Rd,e},F_{b,Rd,i},0.8 \cdot F_{v,Rd} \cdot \beta_{LF})$
$= min\ (270.0,218.9,0.8 \cdot 94.0 \cdot 0.9875)$
$= 74.3\ kN$

$Shear\ capacity = F_{b,Rd} \cdot n_1 \cdot n_2 \cdot N$
$= 74.3 \cdot 6 \cdot 1 \cdot 2$
$= 891.1\ kN$

Applied member shear, $V_{Ed} = 450.0\ kN$

$Unity = \frac{V_{Ed}}{Shear\ capacity}$
$= \frac{450.0}{891.1}$
$= 0.504994$

$891.1\ kN \geq 450.0\ kN$ **(OK)**

$0.505 \leq 1$ **(OK)**

Connection-block tearing of OSL(s) (252). Reference 3.10.2, Part 1-8

Row edge distance top, $e_{1,top} = 40.0\ mm$
Column edge distance, $e_2 = 41.0\ mm$
Bolt rows, $n_1 = 6$
Bolt row spacing, $p_1 = 70\ mm$
Bolt columns, $n_2 = 1$
Column bolt spacing, $p_2 = 70.0\ mm$
Connection thickness, $t = 10.0\ mm$
Tensile strength, $f_u = 430.0\ MPa$
Yield stress, $f_y = 275.0\ MPa$
Bolt row spacing, $p_1 = 70.0\ mm$
Bolt row spacing, $p_1 = 70.0\ mm$

Calculate block shear capacity of each side

Hole diameter, $d_0 = 22.0\ mm$
Hole length, $l_h = 22.0\ mm$
Total length of bolt group, $p_{1,total} = 350.0\ mm$
Gross shear area, $A_{gv} = t \cdot (p_{1,total} + e_{1,top})$
$= 10.0 \cdot (350.0 + 40.0)$
$= 3900.0\ mm^2$
Net shear area, $A_{nv} = t \cdot (p_{1,total} + e_{1,top}) - t \cdot (n_1 - 0.5) \cdot d_0$
$= 10.0 \cdot (350.0 + 40.0) - 10.0 \cdot (6 - 0.5) \cdot 22.0$
$= 2690.0\ mm^2$
Gross tensile area, $A_{gt} = t \cdot (p_2 \cdot (n_2 - 1) + e_2)$
$= 10.0 \cdot (70.0 \cdot (1 - 1) + 41.0)$
$= 410.0\ mm^2$
Net tensile area, $A_{net} = t \cdot (p_2 \cdot (n_2 - 1) + e_2) - t \cdot (n_2 - 0.5) \cdot l_h$
$= 10.0 \cdot (70.0 \cdot (1 - 1) + 41.0) - 10.0 \cdot (1 - 0.5) \cdot 22.0$
$= 300.0\ mm^2$
$\gamma_{M2,1\text{-}1} = 1.1$
$\gamma_{M0} = 1$

Block shear capacity of one side of connection, $V_{Rd,b} = \dfrac{\left(\dfrac{f_u \cdot A_{net}}{\gamma_{M2,1\text{-}1}} + \dfrac{f_y \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$

$$= \frac{\left(\dfrac{430.0 \cdot 300.0}{1.1} + \dfrac{275.0 \cdot 2690.0}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$= 544.4\ kN$

$Shear\ capacity = 2 \cdot V_{Rd,b}$

Connection-block tearing of OSL(s) (252). Reference 3.10.2, Part 1-8 (continued)

$= 2 \cdot 544.4$

$= 1088.7\ kN$

Applied member shear, $V_{Ed} = 450.0\ kN$

$Unity = \dfrac{V_{Ed}}{Shear\ capacity}$

$= \dfrac{450.0}{1088.7}$

$= 0.413337$

$1088.7\ kN \geq 450.0\ kN$ **(OK)**

$0.413 \leq 1$ **(OK)**

Tying resistance-angles and bolts (373). Reference MSB05, 4.3.1.1

Connection tensile strength, $f_u = 430.0\ MPa$

Web thickness, $t_w = 9.0\ mm$

Connection radius, $r = 11.0\ mm$

Connection thickness, $t = 10.0\ mm$

Row edge distance, $e_1 = 40.0\ mm$

Minimum column edge distance, $e_{min} = 41.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt gage, $p_3 = 108.0\ mm$

Bolt rows, $n_1 = 6$

$m = \dfrac{(p_3 - t_w - 2 \cdot t - 2 \cdot 0.8 \cdot r)}{2}$

$= \dfrac{(108.0 - 9.0 - 2 \cdot 10.0 - 2 \cdot 0.8 \cdot 11.0)}{2}$

$= 30.7\ mm$

$n = min\ (e_{min}, 1.25 \cdot m)$

$= min\ (41.0, 1.25 \cdot 30.7)$

$= 38.4\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$e_{1,A} = min\ \left(e_1, 0.5 \cdot (p_3 - t_w - 2 \cdot r) + \dfrac{d_0}{2}\right)$

$= min\ \left(40.0, 0.5 \cdot (108.0 - 9.0 - 2 \cdot 11.0) + \dfrac{22.0}{2}\right)$

$= 40.0\ mm$

$p_{1A,max} = p_3 - t_w - 2 \cdot r + d_0$

$= 108.0 - 9.0 - 2 \cdot 11.0 + 22.0$

$= 99.0\ mm$

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Effective length of the bolt group, $l_{bg,eff} = min\ (p_{1,total}, (n_1 - 1) \cdot p_{1A,max})$

$= min\ (350.0, (6 - 1) \cdot 99.0)$

Tying resistance-angles and bolts (373). Reference MSB05, 4.3.1.1 (continued)

$= 350.0\ mm$

Effective length of a plastic hinge, $\Sigma l_{eff} = 2 \cdot e_{1,A} + l_{bg,eff}$

$= 2 \cdot 40.0 + 350.0$

$= 430.0\ mm$

Width across points of the bolt head or nut, $d_w = 33.0\ mm$

$e_w = \frac{d_w}{4}$

$= \frac{33.0}{4}$

$= 8.2\ mm$

Bolt tension area, $A_{b,t} = 244.8\ mm^2$

$k_2 = 0.9$

Bolt ultimate tensile strength, $f_{ub} = 800.0\ MPa$

$\gamma_{Mu} = 1.1$

Design tension resistance of a bolt, $F_{t,Rd,u} = \frac{\left(\frac{k_2 \cdot f_{ub} \cdot A_{b,t}}{\gamma_{Mu}}\right)}{1000.0}$

$= \frac{\left(\frac{0.9 \cdot 800.0 \cdot 244.8}{1.1}\right)}{1000.0}$

$= 160.2\ kN$

$\gamma_{Mu} = 1.1$

$M_{pl,1,Rd,u} = \frac{\left(\frac{0.25 \cdot \Sigma l_{eff} \cdot t^2 \cdot f_u}{\gamma_{Mu}}\right)}{1000000.0}$

$= \frac{\left(\frac{0.25 \cdot 430.0 \cdot 10.0^2 \cdot 430.0}{1.1}\right)}{1000000.0}$

$= 4.2\ kN \cdot m$

$M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$

$= 4.2\ kN \cdot m$

$F_{Rd,u,1} = \left(\frac{(8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd,u}}{(2 \cdot m \cdot n - e_w \cdot (m + n))}\right) \cdot 1000.0$

$= \left(\frac{(8 \cdot 38.4 - 2 \cdot 8.2) \cdot 4.2}{(2 \cdot 30.7 \cdot 38.4 - 8.2 \cdot (30.7 + 38.4))}\right) \cdot 1000.0$

$= 683.4\ kN$

Tying resistance-angles and bolts (373). Reference MSB05, 4.3.1.1 (continued)

$$F_{Rd,u,2} = \left(\frac{\left(2 \cdot M_{pl,2,Rd,u} + \frac{n \cdot 2 \cdot n_1 \cdot F_{t,Rd,u}}{1000.0}\right)}{(m + n)} \right) \cdot 1000.0$$

$$= \left(\frac{\left(2 \cdot 4.2 + \frac{38.4 \cdot 2 \cdot 6 \cdot 160.2}{1000.0}\right)}{(30.7 + 38.4)} \right) \cdot 1000.0$$

$$= 1189.9\ kN$$

$$F_{Rd,u,3} = 2 \cdot n_1 \cdot F_{t,Rd,u}$$

$$= 2 \cdot 6 \cdot 160.2$$

$$= 1922.7\ kN$$

$$Tying\ capacity = min\ (F_{Rd,u,1}, F_{Rd,u,2}, F_{Rd,u,3})$$

$$= min\ (683.4, 1189.9, 1922.7)$$

$$= 683.4\ kN$$

Applied web tying load, $T_{w,SI,Ed} = 370.0\ kN$

$$Unity = \frac{T_{w,SI,Ed}}{Tying\ capacity}$$

$$= \frac{370.0}{683.4}$$

$$= 0.541411$$

$683.4\ kN \geq 370.0\ kN$ **(OK)**

$0.541 \leq 1$ **(OK)**

Tying resistance-angle gross/net tension (196). Reference 6.2.3, Part 1-1

Bolt rows, $n_1 = 6$

Tensile strength, $f_u = 430.0\ MPa$

Yield stress, $f_y = 275.0\ MPa$

Connection thickness, $t = 10.0\ mm$

Connection depth, $h = 430.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

Gross tensile area, $A_{gt} = 2 \cdot h \cdot t$

$$= 2 \cdot 430.0 \cdot 10.0$$

$$= 8600.0\ mm^2$$

Net tensile area, $A_{net} = 2 \cdot t \cdot (h - n_1 \cdot d_0)$

$$= 2 \cdot 10.0 \cdot (430.0 - 6 \cdot 22.0)$$

$$= 5960.0\ mm^2$$

$\gamma_{Mu} = 1.1$

$\gamma_{M0} = 1$

Tying resistance-angle gross/net tension (196). Reference 6.2.3, Part 1-1 (continued)

$$Tying\ capacity = \frac{min\left(\frac{f_y \cdot A_{gt}}{\gamma_{M0}}, \frac{0.9 \cdot f_u \cdot A_{net}}{\gamma_{Mu}}\right)}{1000.0}$$

$$= \frac{min\left(\frac{275.0 \cdot 8600.0}{1}, \frac{0.9 \cdot 430.0 \cdot 5960.0}{1.1}\right)}{1000.0}$$

$$= 2096.8\ kN$$

Applied web tying load, $T_{w,SI,Ed} = 370.0\ kN$

$$Unity = \frac{T_{w,SI,Ed}}{Tying\ capacity}$$

$$= \frac{370.0}{2096.8}$$

$$= 0.17646$$

$2096.8\ kN \geq 370.0\ kN$ **(OK)**

$0.176 \leq 1$ **(OK)**

Tying resistance-block shear of angles (412). Reference MSB05, 4.3.1.4

Yield stress, $f_y = 275.0\ MPa$

Tensile strength, $f_u = 430.0\ MPa$

Thickness, $t = 10.0\ mm$

Row edge distance, $e_1 = 40.0\ mm$

Column edge distance, $e_2 = 40.0\ mm$

Bolt column spacing, $p_2 = 0.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt columns, $n_2 = 1$

Bolt rows, $n_1 = 6$

C-shaped failure pattern

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Total length of bolt group, $p_{2,total} = 0.0\ mm$

Connection length, $h_p = p_{2,total}$

$= 0.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

Gross tensile area, $A_{gt} = 1 \cdot p_{1,total} \cdot t$

$= 1 \cdot 350.0 \cdot 10.0$

$= 3500.0\ mm^2$

Net tensile area, $A_{net} = A_{gt} - 1 \cdot (n_1 - 1) \cdot d_0 \cdot t$

$= 3500.0 - 1 \cdot (6 - 1) \cdot 22.0 \cdot 10.0$

$= 2400.0\ mm^2$

Gross shear area, $A_{gv} = 1 \cdot 2 \cdot (h_p + e_2) \cdot t$

C-shaped failure pattern (continued)

$= 1 \cdot 2 \cdot (0.0 + 40.0) \cdot 10.0$

$= 800.0\ mm^2$

Net shear area, $A_{nv} = A_{gv} - 1 \cdot 2 \cdot (n_2 - 0.5) \cdot d_0 \cdot t$

$= 800.0 - 1 \cdot 2 \cdot (1 - 0.5) \cdot 22.0 \cdot 10.0$

$= 580.0\ mm^2$

$\gamma_{Mu} = 1.1$

$\gamma_{M0} = 1$

Block shear strength (C-shaped pattern), $F_{Rd,b,C} = \dfrac{\left(\dfrac{f_u \cdot A_{net}}{\gamma_{Mu}} + \dfrac{f_y \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{430.0 \cdot 2400.0}{1.1} + \dfrac{275.0 \cdot 580.0}{\sqrt{3} \cdot 1}\right)}{1000.0}$

$= 1030.3\ kN$

L-shaped failure pattern

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Total length of bolt group, $p_{2,total} = 0.0\ mm$

Connection length, $h_p = p_{2,total}$

$= 0.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

Gross tensile area, $A_{gt} = 1 \cdot (p_{1,total} + e_1) \cdot t$

$= 1 \cdot (350.0 + 40.0) \cdot 10.0$

$= 3900.0\ mm^2$

Net tensile area, $A_{net} = A_{gt} - 1 \cdot (n_1 - 0.5) \cdot d_0 \cdot t$

$= 3900.0 - 1 \cdot (6 - 0.5) \cdot 22.0 \cdot 10.0$

$= 2690.0\ mm^2$

Gross shear area, $A_{gv} = 1 \cdot (h_p + e_2) \cdot t$

$= 1 \cdot (0.0 + 40.0) \cdot 10.0$

$= 400.0\ mm^2$

Net shear area, $A_{nv} = A_{gv} - 1 \cdot (n_2 - 0.5) \cdot d_0 \cdot t$

$= 400.0 - 1 \cdot (1 - 0.5) \cdot 22.0 \cdot 10.0$

$= 290.0\ mm^2$

$\gamma_{Mu} = 1.1$

$\gamma_{M0} = 1$

L-shaped failure pattern (continued)

Block shear strength (L-shaped pattern), $F_{Rd,b,L} = \dfrac{\left(0.5 \cdot \left(\dfrac{f_u \cdot A_{net}}{\gamma_{Mu}}\right) + \dfrac{f_y \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$

$= \dfrac{\left(0.5 \cdot \left(\dfrac{430.0 \cdot 2690.0}{1.1}\right) + \dfrac{275.0 \cdot 290.0}{\sqrt{3} \cdot 1}\right)}{1000.0}$

$= 571.8\ kN$

$Tying\ capacity = 2 \cdot min\ (F_{Rd,b,L}, F_{Rd,b,C})$
$= 2 \cdot min\ (571.8, 1030.3)$
$= 1143.6\ kN$

Applied member shear, $V_{Ed} = 450.0\ kN$

$Unity = \dfrac{V_{Ed}}{Tying\ capacity}$
$= \dfrac{450.0}{1143.6}$
$= 0.393495$

$1143.6\ kN \geq 370.0\ kN$ **(OK)**

$0.393 \leq 1$ **(OK)**

Tying resistance-web leg bolt shear (1). Reference MSB05, 4.3.1.2

Number of shear planes, $N_s = 2$

Bolt columns, $n_2 = 1$

Bolt rows, $n_1 = 6$

Tensile stress area of the bolt, $A = 244.8\ mm^2$

$\gamma_{Mu} = 1.1$

$(t_{pa} = 0.0\ mm) \leq \left(\dfrac{d}{3} = \dfrac{20.0}{3} = 6.7\ mm\right)$

Not necessary to reduce shear capacity for packing.

Bolt shear resistance, $F_{v,Rd} = \dfrac{\left(\dfrac{480.0 \cdot A}{\gamma_{Mu}}\right) \cdot N_s}{1000.0}$

$= \dfrac{\left(\dfrac{480.0 \cdot 244.8}{1.1}\right) \cdot 2}{1000.0}$

$= 213.6\ kN$

$Tying\ capacity = F_{v,Rd} \cdot n_1 \cdot n_2$
$= 213.6 \cdot 6 \cdot 1$
$= 1281.8\ kN$

Applied web tying load, $T_{w,SI,Ed} = 370.0\ kN$

Tying resistance-web leg bolt shear (1). Reference MSB05, 4.3.1.2 (continued)

$$Unity = \frac{T_{w,SI,Ed}}{Tying\ capacity} = \frac{370.0}{1281.8} = 0.288657$$

$1281.8\ kN \geq 370.0\ kN$ **(OK)**

$0.289 \leq 1$ **(OK)**

Tying resistance-bolt bearing on angles (110). Reference MSB05, 4.3.1.3

Tensile strength, $f_u = 430.0\ MPa$

Connection thickness, $t = 10.0\ mm$

Column edge distance, $e_2 = 40.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Row edge distance, $e_1 = 40.0\ mm$

Bolt diameter, $d = 20.0\ mm$

Number of sides, $N = 2$

Bolt columns, $n_2 = 1$

Bolt rows, $n_1 = 6$

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Length of joint, $l_j = p_{1,total} = 350.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

No inner bolts exist in this load direction.

End bolt resistance

$$\alpha_d = \frac{e_2}{3 \cdot d_0} = \frac{40.0}{3 \cdot 22.0} = 0.606061$$

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right) = min\left(0.606061, \frac{800.0}{430.0}, 1\right) = 0.606061$$

$$k_1 = min\left(\frac{2.8 \cdot e_1}{d_0} - 1.7, \frac{1.4 \cdot p_1}{d_0} - 1.7, 2.5\right) = min\left(\frac{2.8 \cdot 40.0}{22.0} - 1.7, \frac{1.4 \cdot 70.0}{22.0} - 1.7, 2.5\right) = 2.5$$

End bolt resistance (continued)

$\gamma_{Mu} = 1.1$

Bearing resistance of end bolt, $F_{b,Rd,e} = \dfrac{\left(\dfrac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{Mu}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{2.5 \cdot 0.606061 \cdot 430.0 \cdot 20.0 \cdot 10.0}{1.1}\right)}{1000.0}$

$= 118.5\ kN$

Bolt shear resistance

Bolt shear resistance, $F_{v,Rd} = 213.6\ kN$

Reduction factor, $\beta_{LF} = 1$

$\left(F_{v,Rd} \cdot \beta_{LF} = 213.6 \cdot 1 = 213.6\ kN\right) \geq \left(F_{b,Rd,e} = 118.5\ kN\right)$

Bolt shear resistance greater than each individual bearing resistance

Bearing resistance, $F_{b,Rd} = F_{b,Rd,e}$

$= 118.5\ kN$

$Tying\ capacity = F_{b,Rd} \cdot n_2 \cdot n_1 \cdot N$

$= 118.5 \cdot 1 \cdot 6 \cdot 2$

$= 1421.5\ kN$

Applied web tying load, $T_{w,SI,Ed} = 370.0\ kN$

$Unity = \dfrac{T_{w,SI,Ed}}{Tying\ capacity}$

$= \dfrac{370.0}{1421.5}$

$= 0.260289$

$1421.5\ kN \geq 370.0\ kN$ **(OK)**

$0.260 \leq 1$ **(OK)**

Tying resistance-beam web tension rupture (124). Reference MSB05, 4.3.2.2

Yield stress, $f_y = 275.0\ MPa$

Tensile strength, $f_u = 430.0\ MPa$

$Web\ thickness = 9.0\ mm$

$Web\ depth = 430.0\ mm$

Bolt rows, $n_1 = 6$

Hole diameter, $d_0 = 22.0\ mm$

Gross plate area, $A_g = Web\ depth \cdot Web\ thickness$

$= 430.0 \cdot 9.0$

$= 3870.0\ mm^2$

Net plate area, $A_n = Web\ thickness \cdot \left(Web\ depth - n_1 \cdot d_0\right)$

$= 9.0 \cdot (430.0 - 6 \cdot 22.0)$

Tying resistance-beam web tension rupture (124). Reference MSB05, 4.3.2.2 (continued)

$= 2682.0\ mm^2$

$\gamma_{Mu} = 1.1$

$\gamma_{M0} = 1$

$$Tying\ capacity = \frac{min\left(\frac{f_y \cdot A_g}{\gamma_{M0}}, \frac{0.9 \cdot f_u \cdot A_n}{\gamma_{Mu}}\right)}{1000.0}$$

$$= \frac{min\left(\frac{275.0 \cdot 3870.0}{1}, \frac{0.9 \cdot 430.0 \cdot 2682.0}{1.1}\right)}{1000.0}$$

$= 943.6\ kN$

Applied member tying load, $T_{SI,Ed} = 370.0\ kN$

$$Unity = \frac{T_{SI,Ed}}{Tying\ capacity}$$

$$= \frac{370.0}{943.6}$$

$= 0.392116$

$943.6\ kN \geq 370.0\ kN$ **(OK)**

$0.392 \leq 1$ **(OK)**

Tying resistance-beam web block shear (412). Reference MSB05, 4.3.2.3

Yield stress, $f_y = 275.0\ MPa$

Tensile strength, $f_u = 430.0\ MPa$

Thickness, $t = 9.0\ mm$

Column edge distance, $e_2 = 40.0\ mm$

Bolt column spacing, $p_2 = 0.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt columns, $n_2 = 1$

Bolt rows, $n_1 = 6$

C-shaped failure pattern

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Total length of bolt group, $p_{2,total} = 0.0\ mm$

Connection length, $h_p = p_{2,total}$

$= 0.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

Gross tensile area, $A_{gt} = 1 \cdot p_{1,total} \cdot t$

$= 1 \cdot 350.0 \cdot 9.0$

$= 3150.0\ mm^2$

Net tensile area, $A_{net} = A_{gt} - 1 \cdot (n_1 - 1) \cdot d_0 \cdot t$

$= 3150.0 - 1 \cdot (6 - 1) \cdot 22.0 \cdot 9.0$

C-shaped failure pattern (continued)

$= 2160.0\ mm^2$

Gross shear area, $A_{gv} = 1 \cdot 2 \cdot (h_p + e_2) \cdot t$

$= 1 \cdot 2 \cdot (0.0 + 40.0) \cdot 9.0$

$= 720.0\ mm^2$

Net shear area, $A_{nv} = A_{gv} - 1 \cdot 2 \cdot (n_2 - 0.5) \cdot d_0 \cdot t$

$= 720.0 - 1 \cdot 2 \cdot (1 - 0.5) \cdot 22.0 \cdot 9.0$

$= 522.0\ mm^2$

$\gamma_{Mu} = 1.1$

$\gamma_{M0} = 1$

Block shear strength (C-shaped pattern), $F_{Rd,b,C} = \dfrac{\left(\dfrac{f_u \cdot A_{net}}{\gamma_{Mu}} + \dfrac{f_y \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$

$$= \frac{\left(\dfrac{430.0 \cdot 2160.0}{1.1} + \dfrac{275.0 \cdot 522.0}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$= 927.2\ kN$

$Tying\ capacity = F_{Rd,b,C}$

$= 927.2\ kN$

Applied web tying load, $T_{w,SI,Ed} = 370.0\ kN$

$Unity = \dfrac{T_{w,SI,Ed}}{Tying\ capacity}$

$= \dfrac{370.0}{927.2}$

$= 0.399051$

$927.2\ kN \geq 370.0\ kN$ **(OK)**

$0.399 \leq 1$ **(OK)**

Tying resistance-bolt bearing on beam web (110). Reference MSB05, 4.3.2.1

Tensile strength, $f_u = 430.0\ MPa$

Plate thickness, $t_p = 9.0\ mm$

Column edge distance, $e_2 = 40.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt diameter, $d = 20.0\ mm$

Bolt columns, $n_2 = 1$

Bolt rows, $n_1 = 6$

Total length of bolt group, $p_{1,total} = 350.0\ mm$

Length of joint, $l_j = p_{1,total}$

$= 350.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Tying resistance-bolt bearing on beam web (110). Reference MSB05, 4.3.2.1 (continued)

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

No inner bolts exist in this load direction.

End bolt resistance

$$\alpha_d = \frac{e_2}{3 \cdot d_0} = \frac{40.0}{3 \cdot 22.0} = 0.606061$$

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right) = min\left(0.606061, \frac{800.0}{430.0}, 1\right) = 0.606061$$

$$k_1 = min\left(\frac{1.4 \cdot p_1}{d_0} - 1.7, 2.5\right) = min\left(\frac{1.4 \cdot 70.0}{22.0} - 1.7, 2.5\right) = 2.5$$

$\gamma_{Mu} = 1.1$

Bearing resistance of end bolt, $F_{b,Rd,e} = \dfrac{\left(\dfrac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t_p}{\gamma_{Mu}}\right)}{1000.0}$

$$= \frac{\left(\frac{2.5 \cdot 0.606061 \cdot 430.0 \cdot 20.0 \cdot 9.0}{1.1}\right)}{1000.0}$$

$$= 106.6\ kN$$

Bolt shear resistance

Bolt shear resistance, $F_{v,Rd} = 213.6\ kN$

Reduction factor, $\beta_{LF} = 1$

$\left(F_{v,Rd} \cdot \beta_{LF} = 213.6 \cdot 1 = 213.6\ kN\right) \geq \left(F_{b,Rd,e} = 106.6\ kN\right)$

Bolt shear resistance greater than each individual bearing resistance

Bearing resistance, $F_{b,Rd} = F_{b,Rd,e}$

$= 106.6\ kN$

$Tying\ capacity = F_{b,Rd} \cdot n_2 \cdot n_1$

$= 106.6 \cdot 1 \cdot 6$

$= 639.7\ kN$

Applied web tying load, $T_{w,SI,Ed} = 370.0\ kN$

Tying resistance-bolt bearing on beam web (110). Reference MSB05, 4.3.2.1 (continued)

$Unity = \frac{T_{w,SI,Ed}}{Tying\ capacity}$

$= \frac{370.0}{639.7}$

$= 0.578397$

$639.7\ kN \geq 370.0\ kN$ **(OK)**

$0.578 \leq 1$ **(OK)**

Bending \& prying of angle(s) (43). Reference SJtE3, Pg 26 \& 79

Distance from bolt c/l to face of tee or c/l of angle leg, $b = 44.5\ mm$

Vertical eccentricity, $e_v = 0.0\ mm$

Web cut, $\gamma = 0.0\ °$

Connection gage, $g_{conn} = 108.0\ mm$

Yield stress, $f_y = 275.0\ MPa$

Clip angle thickness, $t_{ang} = 10.0\ mm$

FS connection length, $L_{fs} = 430.0\ mm$

NS connection length, $L_{ns} = 430.0\ mm$

Bolt diameter, $d = 20.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

FS bolt rows, $n_{1,FS} = 6$

NS bolt rows, $n_{1,NS} = 6$

Applied web tying load, $T_{w,SI,Ed} = 370.0\ kN$

Bolt row spacing, $p_1 = 70.0\ mm$

Max tributary length per bolt, $P_{max} = p_1$

$= 70.0\ mm$

Tributary length per pair of bolts parallel to stem or leg, $P = min\left(\frac{max\ (L_{ns},L_{fs})}{n_{1,NS}}, P_{max}\right)$

$= min\left(\frac{max\ (430.0,430.0)}{6}, 70.0\right)$

$= 70.0\ mm$

Width of bolt hole parallel to tee stem or angle leg, $d' = 22.0\ mm$

$b' = b - 0.5 \cdot d$

$= 44.5 - 0.5 \cdot 20.0$

$= 34.5\ mm$

Ratio of net area (at bolt line) to gross area (at leg of angle), $\delta = 1 - \frac{d'}{P}$

$= 1 - \frac{22.0}{70.0}$

$= 0.685714$

$\gamma_{M0} = 1$

Bending \& prying of angle(s) (43). Reference SJtE3, Pg 26 \& 79 (continued)

Tributary moment per bolt, $m = \dfrac{\left(\dfrac{\left(\dfrac{P \cdot t_{ang} \cdot t_{ang} \cdot f_y}{\gamma_{M0}}\right)}{4}\right)}{1000000.0}$

$= \dfrac{\left(\dfrac{\left(\dfrac{70.0 \cdot 10.0 \cdot 10.0 \cdot 275.0}{1}\right)}{4}\right)}{1000000.0}$

$= 0.5\ kN \cdot m$

Allowable bolt tension, $T_{allow} = \left(\dfrac{(1 + \delta) \cdot m}{b'}\right) \cdot 1000.0$

$= \left(\dfrac{(1 + 0.685714) \cdot 0.5}{34.5}\right) \cdot 1000.0$

$= 23.5\ kN$

Moment of inertia for the bolt group

Bolt row spacing, $p_1 = 70.0\ mm$

NS Bolt group moment of inertia about the major axis, $I_{y,ns} = \dfrac{1 \cdot n_{1,NS} \cdot p_1^2 \cdot (n_{1,NS}^2 - 1)}{12}$

$= \dfrac{1 \cdot 6 \cdot 70.0^2 \cdot (6^2 - 1)}{12}$

$= 85750.0\ mm^2$

Bolt row spacing, $p_1 = 70.0\ mm$

FS Bolt group moment of inertia about the major axis, $I_{y,fs} = \dfrac{1 \cdot n_{1,FS} \cdot p_1^2 \cdot (n_{1,FS}^2 - 1)}{12}$

$= \dfrac{1 \cdot 6 \cdot 70.0^2 \cdot (6^2 - 1)}{12}$

$= 85750.0\ mm^2$

NS Bolt group moment of inertia about the minor axis, $I_{z,ns} = 0.0\ mm^2$

FS Bolt group moment of inertia about the minor axis, $I_{z,fs} = 0.0\ mm^2$

Centroid of NS bolt group along major axis, $\bar{y}_{ns} = 0.0\ mm$

Centroid of FS bolt group along major axis, $\bar{y}_{fs} = 0.0 + (1 - 1) \cdot 0.0 + g_{conn}$

$= 0.0 + (1 - 1) \cdot 0.0 + 108.0$

$= 108.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Moment of inertia for the bolt group (continued)

Centroid of NS bolt group along minor axis, $\bar{z}_{ns} = 175.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Centroid of FS bolt group along minor axis, $\bar{z}_{fs} = 175.0\ mm$

Location of N.A. along major axis, $\bar{y} = \dfrac{(1 \cdot n_{1,NS} \cdot \bar{y}_{ns} + 1 \cdot n_{1,FS} \cdot \bar{y}_{fs})}{(1 \cdot n_{1,NS} + 1 \cdot n_{1,FS})}$

$= \dfrac{(1 \cdot 6 \cdot 0.0 + 1 \cdot 6 \cdot 108.0)}{(1 \cdot 6 + 1 \cdot 6)}$

$= 54.0\ mm$

Location of N.A. along minor axis, $\bar{y} = \dfrac{(1 \cdot n_{1,NS} \cdot \bar{z}_{ns} + 1 \cdot n_{1,FS} \cdot \bar{z}_{fs})}{(1 \cdot n_{1,NS} + 1 \cdot n_{1,FS})}$

$= \dfrac{(1 \cdot 6 \cdot 175.0 + 1 \cdot 6 \cdot 175.0)}{(1 \cdot 6 + 1 \cdot 6)}$

$= 175.0\ mm$

$\delta_{x,\ ns} = |\bar{y}_{ns} - \bar{y}|$

$= |0.0 - 54.0|$

$= 54.0\ mm$

$\delta_{x,\ fs} = |\bar{y}_{fs} - \bar{y}|$

$= |108.0 - 54.0|$

$= 54.0\ mm$

$\delta_{y,\ ns} = |\bar{z}_{ns} - \bar{y}|$

$= |175.0 - 175.0|$

$= 0.0\ mm$

$\delta_{y,\ fs} = |\bar{z}_{fs} - \bar{y}|$

$= |175.0 - 175.0|$

$= 0.0\ mm$

Bolt group moment of inertia about the major axis, $I_y = I_{y,ns} + 1 \cdot n_{1,NS} \cdot \delta^2_{y,\ ns} + I_{y,fs} + 1 \cdot n_{1,FS} \cdot \delta^2_{y,\ fs}$

$= 85750.0 + 1 \cdot 6 \cdot 0.0^2 + 85750.0 + 1 \cdot 6 \cdot 0.0^2$

$= 171500.0\ mm^2$

Bolt group moment of inertia about the minor axis, $I_z = I_{z,ns} + 1 \cdot n_{1,NS} \cdot \delta^2_{x,\ ns} + I_{z,fs} + 1 \cdot n_{1,FS} \cdot \delta^2_{x,\ fs}$

$= 0.0 + 1 \cdot 6 \cdot 54.0^2 + 0.0 + 1 \cdot 6 \cdot 54.0^2$

$= 34992.0\ mm^2$

NS elastic section modulus about the major axis, $S_{y,ns} = \dfrac{I_y}{y}$

$= \dfrac{171500.0}{175.0}$

$= 980.0\ mm$

FS elastic section modulus about the major axis, $S_{y,fs} = \dfrac{I_y}{y}$

$= \dfrac{171500.0}{175.0}$

$= 980.0\ mm$

Bending \& prying of angle(s) (43). Reference SJtE3, Pg 26 \& 79 (continued)

NS elastic section modulus about the minor axis, $S_{z,ns} = \frac{I_z}{y}$

$= \frac{34992.0}{54.0}$
$= 648.0\ mm$

FS elastic section modulus about the minor axis, $S_{z,fs} = \frac{I_z}{(g_{conn} - y)}$

$= \frac{34992.0}{(108.0 - 54.0)}$
$= 648.0\ mm$

Maximum tension on a NS bolt due to tension, $T_{ns} = T_{w,SI,Ed} \cdot \left(\frac{1}{(n_{1,NS} + n_{1,FS})} + \frac{|e_v|}{S_{y,ns}} + \frac{e_h}{S_{z,ns}}\right)$

$= 370.0 \cdot \left(\frac{1}{(6 + 6)} + \frac{|0.0|}{980.0} + \frac{0.0}{648.0}\right)$
$= 30.8\ kN$

Maximum tension on a FS bolt due to tension, $T_{fs} = T_{w,SI,Ed} \cdot \left(\left(\frac{1}{(n_{1,NS} + n_{1,FS})} + \frac{|e_v|}{S_{y,fs}}\right) - \frac{e_h}{S_{z,fs}}\right)$

$= 370.0 \cdot \left(\left(\frac{1}{(6 + 6)} + \frac{|0.0|}{980.0}\right) - \frac{0.0}{648.0}\right)$
$= 30.8\ kN$

Maximum bolt tension caused by load, $T_{max} = max\ (T_{ns}, T_{fs})$
$= max\ (30.8, 30.8)$
$= 30.8\ kN$

$Unity = \frac{T_{max} \cdot cos\ (|\gamma|)}{T_{allow}}$
$= \frac{30.8 \cdot cos\ (|0.0|)}{23.5}$
$= 1.31125$

Calculate maximum capacity

Tried $T_{unit} = 0.4 \cdots\ 282.0 \cdot kN.$

Tension force, $T_{unit} = 282.2\ kN$

Horizontal eccentricity, $e_h = 0.0\ mm$

Moment of inertia for the bolt group

Bolt row spacing, $p_1 = 70.0\ mm$

NS Bolt group moment of inertia about the major axis, $I_{y,ns} = \frac{1 \cdot n_{1,NS} \cdot p_1^2 \cdot (n_{1,NS}^2 - 1)}{12}$

$= \frac{1 \cdot 6 \cdot 70.0^2 \cdot (6^2 - 1)}{12}$
$= 85750.0\ mm^2$

Bolt row spacing, $p_1 = 70.0\ mm$

Moment of inertia for the bolt group (continued)

FS Bolt group moment of inertia about the major axis, $I_{y,fs} = \dfrac{1 \cdot n_{1,FS} \cdot p_1^2 \cdot (n_{1,FS}^2 - 1)}{12}$

$= \dfrac{1 \cdot 6 \cdot 70.0^2 \cdot (6^2 - 1)}{12}$

$= 85750.0\ mm^2$

NS Bolt group moment of inertia about the minor axis, $I_{z,ns} = 0.0\ mm^2$

FS Bolt group moment of inertia about the minor axis, $I_{z,fs} = 0.0\ mm^2$

Centroid of NS bolt group along major axis, $\bar{y}_{ns} = 0.0\ mm$

Centroid of FS bolt group along major axis, $\bar{y}_{fs} = 0.0 + (1 - 1) \cdot 0.0 + g_{conn}$

$= 0.0 + (1 - 1) \cdot 0.0 + 108.0$

$= 108.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Centroid of NS bolt group along minor axis, $\bar{z}_{ns} = 175.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Centroid of FS bolt group along minor axis, $\bar{z}_{fs} = 175.0\ mm$

Location of N.A. along major axis, $\bar{y} = \dfrac{(1 \cdot n_{1,NS} \cdot \bar{y}_{ns} + 1 \cdot n_{1,FS} \cdot \bar{y}_{fs})}{(1 \cdot n_{1,NS} + 1 \cdot n_{1,FS})}$

$= \dfrac{(1 \cdot 6 \cdot 0.0 + 1 \cdot 6 \cdot 108.0)}{(1 \cdot 6 + 1 \cdot 6)}$

$= 54.0\ mm$

Location of N.A. along minor axis, $\bar{y} = \dfrac{(1 \cdot n_{1,NS} \cdot \bar{z}_{ns} + 1 \cdot n_{1,FS} \cdot \bar{z}_{fs})}{(1 \cdot n_{1,NS} + 1 \cdot n_{1,FS})}$

$= \dfrac{(1 \cdot 6 \cdot 175.0 + 1 \cdot 6 \cdot 175.0)}{(1 \cdot 6 + 1 \cdot 6)}$

$= 175.0\ mm$

$\delta_{x,\ ns} = |\bar{y}_{ns} - \bar{y}|$

$= |0.0 - 54.0|$

$= 54.0\ mm$

$\delta_{x,\ fs} = |\bar{y}_{fs} - \bar{y}|$

$= |108.0 - 54.0|$

$= 54.0\ mm$

$\delta_{y,\ ns} = |\bar{z}_{ns} - \bar{y}|$

$= |175.0 - 175.0|$

$= 0.0\ mm$

$\delta_{y,\ fs} = |\bar{z}_{fs} - \bar{y}|$

$= |175.0 - 175.0|$

$= 0.0\ mm$

Bolt group moment of inertia about the major axis, $I_y = I_{y,ns} + 1 \cdot n_{1,NS} \cdot \delta_{y,\ ns}^2 + I_{y,fs} + 1 \cdot n_{1,FS} \cdot \delta_{y,\ fs}^2$

$= 85750.0 + 1 \cdot 6 \cdot 0.0^2 + 85750.0 + 1 \cdot 6 \cdot 0.0^2$

Moment of inertia for the bolt group (continued)

$= 171500.0\ mm^2$

Bolt group moment of inertia about the minor axis, $I_z = I_{z,ns} + 1 \cdot n_{1,NS} \cdot \delta_{x,\ ns}^2 + I_{z,fs} + 1 \cdot n_{1,FS} \cdot \delta_{x,\ fs}^2$

$= 0.0 + 1 \cdot 6 \cdot 54.0^2 + 0.0 + 1 \cdot 6 \cdot 54.0^2$

$= 34992.0\ mm^2$

NS elastic section modulus about the major axis, $S_{y,ns} = \frac{I_y}{y}$

$= \frac{171500.0}{175.0}$

$= 980.0\ mm$

FS elastic section modulus about the major axis, $S_{y,fs} = \frac{I_y}{y}$

$= \frac{171500.0}{175.0}$

$= 980.0\ mm$

NS elastic section modulus about the minor axis, $S_{z,ns} = \frac{I_z}{y}$

$= \frac{34992.0}{54.0}$

$= 648.0\ mm$

FS elastic section modulus about the minor axis, $S_{z,fs} = \frac{I_z}{\left(g_{conn} - \bar{y}\right)}$

$= \frac{34992.0}{\left(108.0 - 54.0\right)}$

$= 648.0\ mm$

Maximum tension on a NS bolt due to tension, $T_{ns} = T_{unit} \cdot \left(\frac{1}{\left(n_{1,NS} + n_{1,FS}\right)} + \frac{|e_v|}{S_{y,ns}} + \frac{e_h}{S_{z,ns}}\right)$

$= 282.2 \cdot \left(\frac{1}{(6 + 6)} + \frac{|0.0|}{980.0} + \frac{0.0}{648.0}\right)$

$= 23.5\ kN$

Maximum tension on a FS bolt due to tension, $T_{fs} = T_{unit} \cdot \left(\left(\frac{1}{\left(n_{1,NS} + n_{1,FS}\right)} + \frac{|e_v|}{S_{y,fs}}\right) - \frac{e_h}{S_{z,fs}}\right)$

$= 282.2 \cdot \left(\left(\frac{1}{(6 + 6)} + \frac{|0.0|}{980.0}\right) - \frac{0.0}{648.0}\right)$

$= 23.5\ kN$

Maximum bolt tension caused by load, $T_{max} = max\ \left(T_{ns}, T_{fs}\right)$

$= max\ (23.5, 23.5)$

$= 23.5\ kN$

Interaction check, $UR = \frac{T_{max} \cdot cos\ (|\gamma|)}{T_{allow}}$

$= \frac{23.5 \cdot cos\ (|0.0|)}{23.5}$

$= 1.00023$

$(UR = 1.00023) \approx 1$

Calculate maximum capacity (continued)

$Tying\ capacity = T_{unit}$

$= 282.2\ kN$

$282.2\ kN < 370.0\ kN$ **(NO GOOD)**

$1.311 > 1$ **(NO GOOD)**

Shear yielding of outstanding leg (15). Reference 6.2.6, Part 1-1

Connection yield stress, $f_y = 275.0\ MPa$

Connection thickness, $t = 10.0\ mm$

Connection depth, $h = 430.0\ mm$

Gross shear area, $A_{gv} = 2 \cdot t \cdot h$

$= 2 \cdot 10.0 \cdot 430.0$

$= 8600.0\ mm^2$

$\gamma_{M0} = 1$

$$Tying\ capacity = \frac{\left(\frac{\left(\frac{f_y \cdot A_{gv}}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1.27}\right)}{1000.0}$$

$$= \frac{\left(\frac{\left(\frac{275.0 \cdot 8600.0}{\sqrt{3} \cdot 1}\right)}{1.27}\right)}{1000.0}$$

$= 1075.1\ kN$

Applied member tying load, $T_{SI,Ed} = 370.0\ kN$

$$Unity = \frac{T_{SI,Ed}}{Tying\ capacity}$$

$$= \frac{370.0}{1075.1}$$

$= 0.344154$

$1075.1\ kN \geq 370.0\ kN$ **(OK)**

$0.344 \leq 1$ **(OK)**

Shear rupture of outstanding leg (21). Reference 6.2, Part 1-1

Connection tensile strength, $f_u = 410.0\ MPa$

Bolt rows, $n_1 = 6$

Connection thickness, $t = 10.0\ mm$

Connection depth, $h = 430.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

Net shear area, $A_{nv} = 2 \cdot t \cdot (h - n_1 \cdot d_0)$

$= 2 \cdot 10.0 \cdot (430.0 - 6 \cdot 22.0)$

$= 5960.0\ mm^2$

$\gamma_{M2,1\text{-}1} = 1.1$

Tying capacity, $T_{SI,Rd} = \dfrac{\left(\dfrac{f_u \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M2,1\text{-}1}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{410.0 \cdot 5960.0}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$

$= 1282.6\ kN$

$Tying\ capacity = T_{SI,Rd}$

$= 1282.6\ kN$

Applied member tying load, $T_{SI,Ed} = 370.0\ kN$

$Unity = \dfrac{T_{SI,Ed}}{Tying\ capacity}$

$= \dfrac{370.0}{1282.6}$

$= 0.288477$

$1282.6\ kN \geq 370.0\ kN$ **(OK)**

$0.288 \leq 1$ **(OK)**

Tying resistance-bending of supporting flange (177). Reference Pg 26 \& 79

Web cut angle, $\gamma = 0.0\ °$

Bolt row spacing, $p_1 = 70\ mm$

FS bolt rows, $n_{1,FS} = 6$

NS bolt rows, $n_{1,NS} = 6$

Supporting member yield stress, $f_{y,2} = 275.0\ MPa$

Column flange gage, $g_{f,col} = 108.0\ mm$

Supporting member flange thickness, $t_{f,s} = 15.7\ mm$

Supporting member web thickness, $t_{w,2} = 9.0\ mm$

Bolt diameter, $d = 20.0\ mm$

Distance from bolt c/l to face of tee or c/l of angle leg, $b = 0.5 \cdot (g_{f,col} - t_{w,2})$

Tying resistance-bending of supporting flange (177). Reference Pg 26 \& 79 (continued)

$= 0.5 \cdot (108.0 - 9.0)$

$= 49.5\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Tributary length per pair of bolts parallel to stem or leg, $P = min\ (p_1, g_{f,col})$

$= min\ (70.0, 108.0)$

$= 70.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Max tributary length per bolt, $P_{max} = p_1$

$= 70.0\ mm$

Width of bolt hole parallel to tee stem or angle leg, $d' = 22.0\ mm$

$b' = b - 0.5 \cdot d$

$= 49.5 - 0.5 \cdot 20.0$

$= 39.5\ mm$

Ratio of net area (at bolt line) to gross area (at leg of angle), $\delta = 1 - \frac{d'}{P}$

$= 1 - \frac{22.0}{70.0}$

$= 0.685714$

$\gamma_{M0} = 1$

Tributary moment per bolt, $m = \frac{\left(\frac{\left(\frac{P \cdot t_{f,s} \cdot t_{f,s} \cdot f_{y,2}}{\gamma_{M0}}\right)}{4}\right)}{1000000.0}$

$= \frac{\left(\frac{\left(\frac{70.0 \cdot 15.7 \cdot 15.7 \cdot 275.0}{1}\right)}{4}\right)}{1000000.0}$

$= 1.2\ kN \cdot m$

Force per bolt, $t = \left(\frac{(1 + \delta) \cdot m}{b'}\right) \cdot 1000.0$

$= \left(\frac{(1 + 0.685714) \cdot 1.2}{39.5}\right) \cdot 1000.0$

$= 50.6\ kN$

Tying capacity $= \frac{t \cdot (n_{1,NS} + n_{1,FS})}{cos\ (|\gamma|)}$

Tying resistance-bending of supporting flange (177). Reference Pg 26 \& 79 (continued)

$$= \frac{50.6 \cdot (6 + 6)}{cos\ (|0.0|)}$$

$$= 607.5\ kN$$

Applied web tying load, $T_{w,SI,Ed} = 370.0\ kN$

$$Unity = \frac{T_{w,SI,Ed}}{Tying\ capacity}$$

$$= \frac{370.0}{607.5}$$

$$= 0.609054$$

$607.5\ kN \geq 370.0\ kN$ **(OK)**

$0.609 \leq 1$ **(OK)**

Results summary

Clip Angle on left end of Beam B_15 [19]

Design calculation summary for member [19], left end

Desc. Ref.:	Calc. Num.	Unity Ratio	Resistance	Design Force	EUROCODE ref.
1:	(3)	0.468	962.0 kN	450.0 kN	Table 3.4, Part 1-8
2:	(20)	0.422	1066.8 kN	450.0 kN	3.6, Part 1-8
3:	(20)	0.773	582.5 kN	450.0 kN	Table 3.4, Part 1-8
4:	(3)	0.499	902.4 kN	450.0 kN	3.6, Part 1-8
5:	(110)	0.505	891.1 kN	450.0 kN	3.6, Part 1-8
6:	(110)	0.505	891.1 kN	450.0 kN	3.6, Part 1-8
7:	(15)	0.419	1075.1 kN	450.0 kN	6.2.6, Part 1-1
8:	(21)	0.335	1345.1 kN	450.0 kN	6.2, Part 1-1
9:	(6)	0.465	967.6 kN	450.0 kN	3.10.2, Part 1-8
10:	(2)	0.470	957.3 kN	450.0 kN	6.2.6, Part 1-1
11:	(4)	0.412	1092.7 kN	450.0 kN	6.2.6, Part 1-1
12:	(373)	0.541	683.4 kN	370.0 kN	MSB05, 4.3.1.1
13:	(196)	0.176	2096.8 kN	370.0 kN	6.2.3, Part 1-1
14:	(1)	0.289	1281.8 kN	370.0 kN	MSB05, 4.3.1.2
15:	(110)	0.260	1421.5 kN	370.0 kN	MSB05, 4.3.1.3
16:	(412)	0.393	1143.6 kN	370.0 kN	MSB05, 4.3.1.4
17:	(110)	0.578	639.7 kN	370.0 kN	MSB05, 4.3.2.1
18:	(124)	0.392	943.6 kN	370.0 kN	MSB05, 4.3.2.2
19:	(412)	0.399	927.2 kN	370.0 kN	MSB05, 4.3.2.3
20:	(43)	1.311	282.2 kN	370.0 kN	SJtE3, Pg 26 & 79
21:	(15)	0.344	1075.1 kN	370.0 kN	6.2.6, Part 1-1
22:	(21)	0.288	1282.6 kN	370.0 kN	6.2, Part 1-1
23:	(177)	0.609	607.5 kN	370.0 kN	Pg 26 & 79

Check number reference

1:	Web leg-bolt shear
2:	Bolts-bearing on web cover plates
3:	Web leg-bearing on web
4:	Outstanding leg-bolt shear
5:	Outstanding leg-bearing on angles
6:	Outstanding leg-bearing on support
7:	Shear yielding of angle(s)
8:	Shear rupture of angle(s)
9:	Connection-block tearing of web leg(s)
10:	Shear yielding of beam web
11:	Supported beam-net web shear
12:	Tying resistance-angles and bolts (SI)
13:	Tying resistance-angle gross/net tension (SI)
14:	Tying resistance-web leg bolt shear (SI)
15:	Tying resistance-bolt bearing on angles (SI)
16:	Tying resistance-block shear of angles (SI)
17:	Tying resistance-bolt bearing on beam web (SI)
18:	Tying resistance-beam web tension rupture (SI)
19:	Tying resistance-beam web block shear (SI)
20:	Bending & prying of angle(s) (SI)
21:	Shear yielding of OSL (SI)
22:	Shear rupture of OSL (SI)
23:	Tying resistance-bending of supporting flange (SI)

Warnings

- FORCED CONNECTION
    - Engineering review required to evaluate strength and the application of SDS2 design calculations to the specific material and geometry.

Notes and conclusions

- Web leg bolt design includes the effect of eccentricity.
- CONNECTION DESIGN FAILURE
    - Allowable clip OSL bending strength exceeded ⚠

Additional notes

Section 4.4 Worked Example: Angle Web Cleats
MSB05_Joint_Design.pdf