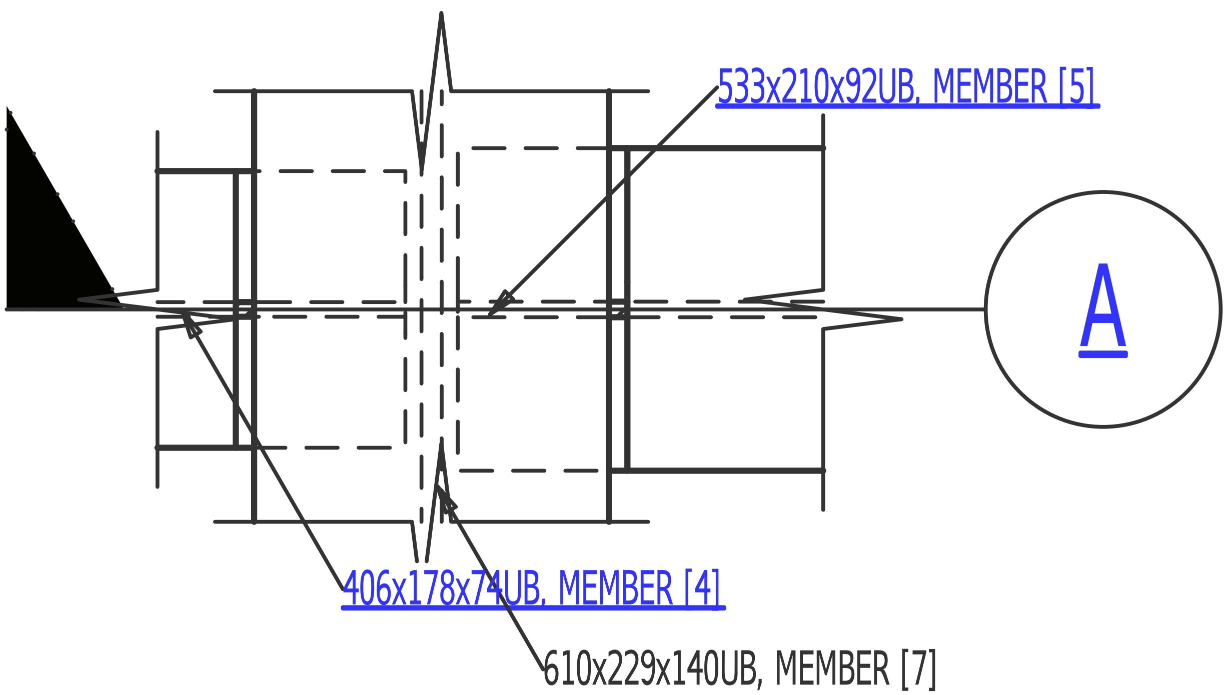
SDS2 Steel Connection Design: Connection Cube Report

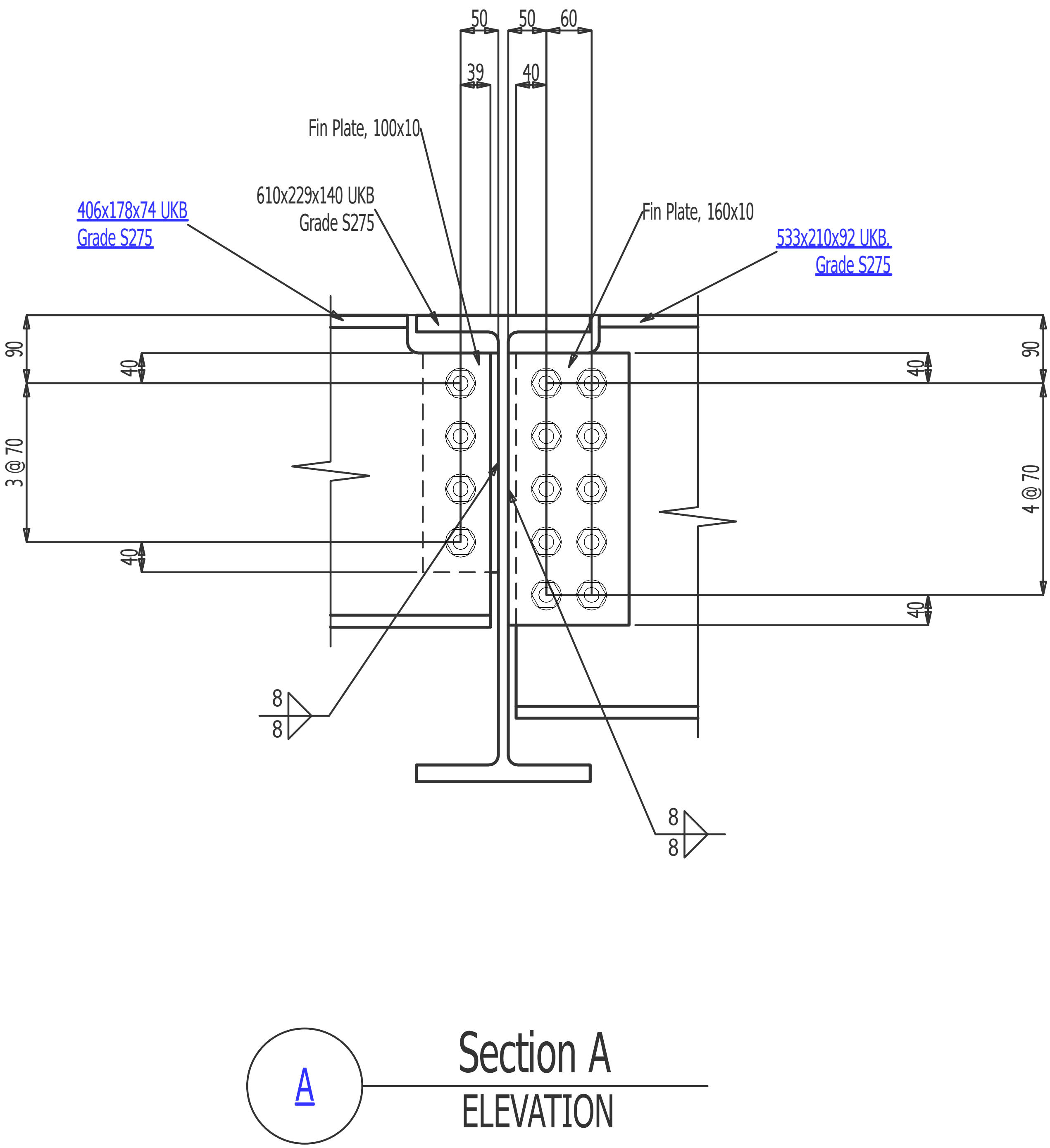
Cube: Fin plates
Revision: 0
Project: 1046_Validation_Eurocode_3UK
Engineer:
Fabricator: Validation_Eurocode_3UK

Generated by SDS2 x 2025.07 on Wednesday, Mar 19, 2025

Fin plates [1] at X=5000, Y=5000 Elev=-267



TOP SIDE VIEW


50
50
60
39
40
Fin Plate, 100x10
610x229x140 UKB
Grade S275
406x178x74 UKB
Grade S275
Fin Plate, 160x10
533x210x92 UKB.
Grade S275
90
40
3 @ 70
40
90
40
4 @ 70
40
8
8
8
8
A
Section A
ELEVATION

Beam B_3 [4]

Design method

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

Overview

Section size:	406x178x74UB
Sequence:	1
ABM:	N/Assign
Plan length:	5000
Camber:	0.00 mm
Span length:	5000
Slope:	0.00 °
Material length:	4983
Plan rotation:	0.00 °

Section properties

Material grade:	S275b
Yield stress, f_y:	275.0 MPa
Tensile strength, f_u:	410.0 MPa
Depth, h:	412.8 mm
Web thickness, t_w:	9.5 mm
Flange width, b:	179.5 mm
Flange thickness, t_f:	16.0 mm
Root radius, r:	10.2 mm
Distance between web toes of fillets, d:	360.4 mm
Moment of inertia about the major axis, I_y:	273.1 10^6 mm^4

Design summary

Right end

Connection:	Shear tab
	Plate, Size as required
	No Stiffener Opposite
	Shear plate on FS, Skew holes in beam
	Combine shear plates: No
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	0
Minus Dim:	17.0 mm (AUTO)
Mtrl Setback:	17.0 mm (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	200.0 kN
Moment:	0.0 kN·m (AUTO)
Tension:	0.0 kN
Compress:	0.0 kN
Tying:	0.0 kN (AUTO)

B_3 [4] Connection strength check: RIGHT END

Member end summary

Connecting nodes

Node 1

Beam:	B_4 [7]
Section size:	610x229x140UB
End 0 elevation:	0 mm
End 1 elevation:	0 mm
Support intersection elevation:	0
Supporting beam rotation:	0.00 degrees
	(looking toward left end)
Material grade:	S275-7
Distance between web toes of fillets, d:	547.6 mm
Supporting member thickness, t_2:	13.1 mm

Node 1 notes

- A member frames to the opposite side of this member

Factored loads

Shear: 200.0 kN

Design load notes

- Reaction has been input
- Design reaction is 30.3 % of the allowable uniform steel beam load.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	S275
	Tensile strength, f_u:	410.0 MPa
	Yield stress, f_y:	275.0 MPa
	Thickness, t:	10.0 mm
	Width, b:	100.0 mm
	Depth, h:	290.0 mm
	Weld line to bolt group c.g., a:	50.4 mm
Weld:	**Weld type:**	Double fillet
	Weld leg size, s:	8.0 mm
	Total effective weld throat, a_e:	11.31 mm
	Weld metal strength, F_{exx}:	0.0 MPa
Bolts:	**Bolt type:**	8.8, Category A
	Hole type in connection:	Standard round
	Bolt diameter, d:	20
	Bolt rows, n_1:	4
	Bolt row spacing, p_1:	70.0 mm
	Bolt columns, n_2:	1
	Web end distance, $e_{1,w}$:	40.0 mm
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit:	1
(at dd) Not Locked:	278

Cope information

Top cope depth, d_{nt}:	50.0 mm
Top cope length, l_{nt}:	110.0 mm

Cope notes

- Cope length dimension is from the end of the beam web.
- At coped section : S_{net} = 324070.50 mm^3, h_o = 362.80 mm
- $e_{2,b}$ = 40.00 mm, $e_{1,b}$ = 40.00 mm

Expanded design calculation

Supported beam-gross web shear (5). Reference Pg 121

Yield stress, $f_y = 275.0\ MPa$

Top cope depth, $d_{nt} = 50.0\ mm$

Flange thickness, $t_f = 16.0\ mm$

Flange width, $b = 179.5\ mm$

Web thickness, $t_w = 9.5\ mm$

Full section depth, $h = 412.8\ mm$

Root radius, $r = 10.2\ mm$

Area of coped section, $A_{Tee} = \left(h - t_f - d_{nt}\right) \cdot t_w + b \cdot t_f + 0.429204 \cdot r^2$

$= \left(412.8 - 16.0 - 50.0\right) \cdot 9.5 + 179.5 \cdot 16.0 + 0.429204 \cdot 10.2^2$

$= 6211.3\ mm^2$

Gross area, $A = A_{Tee} - b \cdot t_f + \left(t_w + 2 \cdot r\right) \cdot \left(\frac{t_f}{2}\right)$

$= 6211.3 - 179.5 \cdot 16.0 + \left(9.5 + 2 \cdot 10.2\right) \cdot \left(\frac{16.0}{2}\right)$

$= 3578.5\ mm^2$

$\gamma_{M0} = 1$

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

$$= \frac{\left(\frac{275.0 \cdot 3578.5}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$= 568.2\ kN$

Applied member shear, $V_{Ed} = 200.0\ kN$

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

$$= \frac{200.0}{568.2}$$

$= 0.351989$

$568.2\ kN \geq 200.0\ kN$ **(OK)**

$0.352 \leq 1$ **(OK)**

Supported beam-net web shear (4). Reference Pg 121

Tensile strength, $f_u = 410.0\ MPa$

Top cope depth, $d_{nt} = 50.0\ mm$

Bolt rows, $n_1 = 4$

Flange thickness, $t_f = 16.0\ mm$

Flange width, $b = 179.5\ mm$

Supported beam-net web shear (4). Reference Pg 121 (continued)

Web thickness, $t_w = 9.5\ mm$

Full section depth, $h = 412.8\ mm$

Root radius, $r = 10.2\ mm$

Area of coped section, $A_{Tee} = \left(h - t_f - d_{nt}\right) \cdot t_w + b \cdot t_f + 0.429204 \cdot r^2$

$= (412.8 - 16.0 - 50.0) \cdot 9.5 + 179.5 \cdot 16.0 + 0.429204 \cdot 10.2^2$

$= 6211.3\ mm^2$

Gross shear area, $A_{v,w} = A_{Tee} - b \cdot t_f + (t_w + 2 \cdot r) \cdot \left(\frac{t_f}{2}\right)$

$= 6211.3 - 179.5 \cdot 16.0 + (9.5 + 2 \cdot 10.2) \cdot \left(\frac{16.0}{2}\right)$

$= 3578.5\ mm^2$

Hole diameter, $d_0 = 22.0\ mm$

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

$= 3578.5 - 4 \cdot 22.0 \cdot 9.5$

$= 2742.5\ mm^2$

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

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

$$= \frac{\left(\frac{410.0 \cdot 2742.5}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$$

$= 590.2\ kN$

Applied member shear, $V_{Ed} = 200.0\ kN$

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

$= \frac{200.0}{590.2}$

$= 0.338868$

$590.2\ kN \geq 200.0\ kN$ **(OK)**

$0.339 \leq 1$ **(OK)**

Supported beam-web block shear (6). Reference Pg 121

Plate thickness, $Web\ thickness = 9.5\ mm$

Yield stress, $f_y = 275.0\ MPa$

Tensile strength, $f_u = 410.0\ MPa$

Bolt column spacing, $p_2 = 0.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt rows, $n_1 = 4$

Supported beam-web block shear (6). Reference Pg 121 (continued)

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, $p_{1,total} = 210.0\ mm$

Gross shear area, $A_{gv} = Web\ thickness \cdot (p_{1,total} + e_1)$

$= 9.5 \cdot (210.0 + 40.0)$

$= 2375.0\ mm^2$

Net shear area, $A_{nv} = Web\ thickness \cdot (p_{1,total} + e_1) - Web\ thickness \cdot (n_1 - 0.5) \cdot d_0$

$= 9.5 \cdot (210.0 + 40.0) - 9.5 \cdot (4 - 0.5) \cdot 22.0$

$= 1643.5\ mm^2$

Gross tensile area, $A_{gt} = Web\ thickness \cdot (p_2 \cdot (n_2 - 1) + e_2)$

$= 9.5 \cdot (0.0 \cdot (1 - 1) + 40.0)$

$= 380.0\ mm^2$

Net tensile area, $A_{net} = Web\ thickness \cdot (p_2 \cdot (n_2 - 1) + e_2) - Web\ thickness \cdot (n_2 - 0.5) \cdot l_h$

$= 9.5 \cdot (0.0 \cdot (1 - 1) + 40.0) - 9.5 \cdot (1 - 0.5) \cdot 22.0$

$= 275.5\ mm^2$

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

$\gamma_{M0} = 1$

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

$$= \frac{\left(0.5 \cdot \left(\frac{410.0 \cdot 275.5}{1.1}\right) + \frac{275.0 \cdot 1643.5}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$= 312.3\ kN$

Applied member shear, $V_{Ed} = 200.0\ kN$

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

$= \frac{200.0}{312.3}$

$= 0.64041$

$312.3\ kN \geq 200.0\ kN$ **(OK)**

$0.640 \leq 1$ **(OK)**

Supported beam-stability at notch (41). Reference Pg 125

Bottom cope length, $l_{nb} = 0.0\ mm$
Bottom cope depth, $d_{nb} = 0.0\ mm$
Top cope length, $l_{nt} = 110.0\ mm$
Top cope depth, $d_{nt} = 50.0\ mm$
Beam depth, $h_b = 412.8\ mm$
Effective top notch depth, $d_{nt,e} = max\ (d_{nt}, 0.0)$
$= max\ (50.0, 0.0)$
$= 50.0\ mm$
Effective top notch length, $l_{nt,e} = l_{nt}$
$= 110.0\ mm$
Effective bottom notch depth, $d_{nb,e} = max\ (d_{nb}, 0.0)$
$= max\ (0.0, 0.0)$
$= 0.0\ mm$
Effective bottom notch length, $l_{nb,e} = l_{nb}$
$= 0.0\ mm$
Top flange is coped.
Reduced beam depth, $h_o = h_b - d_{nt,e} - d_{nb,e}$
$= 412.8 - 50.0 - 0.0$
$= 362.8\ mm$

$$\left(d_{nt,e} = 50.0\ mm\right) \leq \left(\frac{h_b}{2} = \frac{412.8}{2} = 206.4\ mm\right)$$

$$\left(\frac{h_b}{t_w} = \frac{412.8}{9.5} = 43.4526\right) \leq 54.3$$

$$\left(l_{nt,e} = 110.0\ mm\right) \leq \left(h_b = 412.8\ mm\right)$$

Basic requirements are met. Local stability does not need to be checked.

Supported beam-bearing on web (20). Reference Pg 117

Column edge distance, $e_2 = 40.0\ mm$
Top vertical edge distance, $e_{1,t} = 40.0\ mm$
Connection thickness, $t = 9.5\ mm$
Connection tensile strength, $f_u = 410.0\ MPa$
Bolt row spacing, $p_1 = 70\ mm$
Bolt columns, $n_2 = 1$
Bolt rows, $n_1 = 4$
Distance from support to center of bolt group, $z = 50.4\ mm$
Bolt diameter, $d = 20.0\ mm$

Calculate bearing resistance in vertical direction

Total length of bolt group, $p_{1,total} = 210.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt row spacing, $p_1 = 70.0\ mm$

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

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right) = min\left(0.810606, \frac{800.0}{410.0}, 1\right) = 0.810606$$

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

$$= \frac{\left(\dfrac{2.5 \cdot 0.810606 \cdot 410.0 \cdot 20.0 \cdot 9.5}{1.25}\right)}{1000.0}$$

$$= 126.3\ kN$$

End bolt resistance

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

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right) = min\left(0.606061, \frac{800.0}{410.0}, 1\right) = 0.606061$$

Bolt column spacing, $p_2 = 0.0\ mm$

End bolt resistance (continued)

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

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

$= 2.5$

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

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

$= \frac{\left(\frac{2.5 \cdot 0.606061 \cdot 410.0 \cdot 20.0 \cdot 9.5}{1.25}\right)}{1000.0}$

$= 94.4\ kN$

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

$= min\left(94.4, 126.3\right)$

$= 94.4\ kN$

Calculate bearing resistance in horizontal direction

Controlling row edge distance in the vertical direction, $e_1 = e_{1,t}$

$= 40.0\ mm$

Total length of bolt group, $p_{1,total} = 210.0\ mm$

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

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

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

$= 0.606061$

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

$= min\left(0.606061, \frac{800.0}{410.0}, 1\right)$

$= 0.606061$

Bolt row spacing, $p_1 = 70.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

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

End bolt resistance (continued)

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

$$= 2.5$$

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

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

$$= \frac{\left(\dfrac{2.5 \cdot 0.606061 \cdot 410.0 \cdot 20.0 \cdot 9.5}{1.25}\right)}{1000.0}$$

$$= 94.4\ kN$$

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

$$= 94.4\ kN$$

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

$$= 4 \cdot 1$$

$$= 4$$

$\alpha = 0$

Bolt row spacing, $p_1 = 70.0\ mm$

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

$$= \frac{6 \cdot 50.4}{4 \cdot (4 + 1) \cdot 70.0}$$

$$= 0.216214$$

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

$$= \frac{4}{\sqrt{\left(\dfrac{(1 + 0 \cdot 4)}{94.4}\right)^2 + \left(\dfrac{0.216214 \cdot 4}{94.4}\right)^2}}$$

$$= 285.7\ kN$$

Applied member shear, $V_{Ed} = 200.0\ kN$

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

$$= \frac{200.0}{285.7}$$

$$= 0.700036$$

$285.7\ kN \geq 200.0\ kN$ **(OK)**

$0.700 \leq 1$ **(OK)**

Supported beam-fin plate block shear (6). Reference Pg 118

Plate thickness, $t_p = 10.0\ mm$

Yield stress, $f_y = 275.0\ MPa$

Tensile strength, $f_u = 410.0\ MPa$

Bolt column spacing, $p_2 = 0.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt rows, $n_1 = 4$

Column edge distance, $e_2 = 49.5\ mm$

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, $p_{1,total} = 210.0\ mm$

Gross shear area, $A_{gv} = t_p \cdot (p_{1,total} + e_1)$

$= 10.0 \cdot (210.0 + 40.0)$

$= 2500.0\ mm^2$

Net shear area, $A_{nv} = t_p \cdot (p_{1,total} + e_1) - t_p \cdot (n_1 - 0.5) \cdot d_0$

$= 10.0 \cdot (210.0 + 40.0) - 10.0 \cdot (4 - 0.5) \cdot 22.0$

$= 1730.0\ mm^2$

Gross tensile area, $A_{gt} = t_p \cdot (p_2 \cdot (n_2 - 1) + e_2)$

$= 10.0 \cdot (0.0 \cdot (1 - 1) + 49.5)$

$= 495.5\ mm^2$

Net tensile area, $A_{net} = t_p \cdot (p_2 \cdot (n_2 - 1) + e_2) - t_p \cdot (n_2 - 0.5) \cdot l_h$

$= 10.0 \cdot (0.0 \cdot (1 - 1) + 49.5) - 10.0 \cdot (1 - 0.5) \cdot 22.0$

$= 385.5\ mm^2$

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

$\gamma_{M0} = 1$

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

$$= \frac{\left(0.5 \cdot \left(\frac{410.0 \cdot 385.5}{1.1}\right) + \frac{275.0 \cdot 1730.0}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$= 346.5\ kN$

Applied member shear, $V_{Ed} = 200.0\ kN$

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

$$= \frac{200.0}{346.5}$$

Supported beam-fin plate block shear (6). Reference Pg 118 (continued)

$= 0.577201$

$346.5\ kN \geq 200.0\ kN$ **(OK)**

$0.577 \leq 1$ **(OK)**

Supported beam-fin plate gross shear (38). Reference Pg 118

Depth, $h = 290.0\ mm$

Plate thickness, $t_p = 10.0\ mm$

Plate yield stress, $f_{y,p} = 275.0\ MPa$

Applied member shear, $V_{Ed} = 200.0\ kN$

Tension force, horizontal component, $N_{h,t,Ed} = 0.0\ kN$

Compression force, horizontal component, $N_{h,c,Ed} = 0.0\ kN$

Gross area, $A = h \cdot t_p$

$= 290.0 \cdot 10.0$

$= 2900.0\ mm^2$

$\gamma_{M0} = 1$

Plate shear resistance, $V_{Rd} = \dfrac{\left(\dfrac{\left(\dfrac{f_{y,p} \cdot A}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1.27}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{\left(\dfrac{275.0 \cdot 2900.0}{\sqrt{3} \cdot 1}\right)}{1.27}\right)}{1000.0}$

$= 362.5\ kN$

$Unity = \dfrac{V_{Ed}}{V_{Rd}}$

$= \dfrac{200.0}{362.5}$

$= 0.551651$

$Shear\ capacity = V_{Rd}$

$= 362.5\ kN$

$362.5\ kN \geq 200.0\ kN$ **(OK)**

$0.552 \leq 1$ **(OK)**

Connection-net shear rupture (21). Reference Pg 118

Connection tensile strength, $f_u = 410.0\ MPa$

Bolt rows, $n_1 = 4$

Plate thickness, $t_p = 10.0\ mm$

Plate depth, $h_p = 290.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

Net shear area, $A_{nv} = t_p \cdot (h_p - n_1 \cdot d_0)$

$= 10.0 \cdot (290.0 - 4 \cdot 22.0)$

$= 2020.0\ mm^2$

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

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

$= \dfrac{\left(\dfrac{410.0 \cdot 2020.0}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$

$= 434.7\ kN$

$Shear\ capacity = V_{Rd}$

$= 434.7\ kN$

Applied member shear, $V_{Ed} = 200.0\ kN$

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

$= \dfrac{200.0}{434.7}$

$= 0.460088$

$434.7\ kN \geq 200.0\ kN$ **(OK)**

$0.460 \leq 1$ **(OK)**

Supported beam-fin plate bending and lateral torsional buckling (19). Reference Pg 119

Plate yield stress, $f_{y,p} = 275.0\ MPa$

Plate thickness, $t_p = 10.0\ mm$

Plate depth, $h_p = 290.0\ mm$

Distance from weld to centroid of bolt group, $z = 50.4\ mm$

Elastic section modulus, $W_{el,p} = \dfrac{t_p \cdot h_p^2}{6}$

$= \dfrac{10.0 \cdot 290.0^2}{6}$

$= 140166.7\ mm^3$

$\gamma_{M0} = 1$

Distance from weld to first column of bolts, $z_p = z$

Supported beam-fin plate bending and lateral torsional buckling (19). Reference Pg 119 (continued)

$= 50.4\ mm$

$\left(z_p = 50.4\ mm\right) \le \left(\frac{t_p}{0.15} = \frac{10.0}{0.15} = 66.7\ mm\right)$

Short Fin Plate

$$Shear\ capacity = \frac{\left(\frac{\left(\frac{W_{el,p}}{z}\right) \cdot f_{y,p}}{\gamma_{M0}}\right)}{1000.0}$$

$$= \frac{\left(\frac{\left(\frac{140166.7}{50.4}\right) \cdot 275.0}{1}\right)}{1000.0}$$

$= 764.0\ kN$

Applied member shear, $V_{Ed} = 200.0\ kN$

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

$= \frac{200.0}{764.0}$

$= 0.26178$

$764.0\ kN \ge 200.0\ kN$ **(OK)**

$0.262 \le 1$ **(OK)**

Supported beam-bolt shear (3). Reference Pg 115

Number of shear planes, $N_s = 1$

Bolt row spacing, $p_1 = 70\ mm$

Bolt columns, $n_2 = 1$

Bolt rows, $n_1 = 4$

Distance from weld to center of bolt group, $z = 50.4\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

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

$= \frac{6 \cdot 50.4}{4 \cdot \left(4 + 1\right) \cdot 70.0}$

$= 0.216214$

Supported beam-bolt shear (3). Reference Pg 115 (continued)

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

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

$$= 94.0\ kN$$

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

$$= \left(\frac{4 \cdot 1}{\sqrt{1^2 + (0.216214 \cdot 4 \cdot 1)^2}}\right) \cdot 94.0$$

$$= 284.4\ kN$$

Applied member shear, $V_{Ed} = 200.0\ kN$

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

$$= \frac{200.0}{284.4}$$

$$= 0.703235$$

$284.4\ kN \geq 200.0\ kN$ **(OK)**

$0.703 \leq 1$ **(OK)**

Supported beam-bearing on fin plate (20). Reference Pg 116

Column edge distance, $e_2 = 49.5\ mm$

Bottom vertical edge distance, $e_{1,b} = 40.0\ mm$

Top vertical edge distance, $e_{1,t} = 40.0\ mm$

Connection thickness, $t = 10.0\ mm$

Connection tensile strength, $f_u = 410.0\ MPa$

Bolt row spacing, $p_1 = 70\ mm$

Bolt columns, $n_2 = 1$

Bolt rows, $n_1 = 4$

Distance from support to center of bolt group, $z = 50.4\ mm$

Bolt diameter, $d = 20.0\ mm$

Calculate bearing resistance in vertical direction

Total length of bolt group, $p_{1,total} = 210.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt row spacing, $p_1 = 70.0\ mm$

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

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right) = min\left(0.810606, \frac{800.0}{410.0}, 1\right) = 0.810606$$

Bolt column spacing, $p_2 = 0.0\ mm$

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right) = min\left(\frac{2.8 \cdot 49.5}{22.0} - 1.7, 2.5\right) = 2.5$$

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

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

$$= \frac{\left(\dfrac{2.5 \cdot 0.810606 \cdot 410.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$$

$$= 132.9\ kN$$

End bolt resistance

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

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right) = min\left(0.606061, \frac{800.0}{410.0}, 1\right) = 0.606061$$

Bolt column spacing, $p_2 = 0.0\ mm$

End bolt resistance (continued)

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

$= min\left(\frac{2.8 \cdot 49.5}{22.0} - 1.7, 2.5\right)$

$= 2.5$

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

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

$= \frac{\left(\frac{2.5 \cdot 0.606061 \cdot 410.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$

$= 99.4\ kN$

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

$= min\ (99.4, 132.9)$

$= 99.4\ kN$

Calculate bearing resistance in horizontal direction

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

$= min\ (40.0, 40.0)$

$= 40.0\ mm$

Total length of bolt group, $p_{1,total} = 210.0\ mm$

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

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

$= \frac{49.5}{3 \cdot 22.0}$

$= 0.750758$

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

$= min\left(0.750758, \frac{800.0}{410.0}, 1\right)$

$= 0.750758$

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, $F_{b,Rd,e} = \dfrac{\left(\dfrac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$$= \frac{\left(\frac{2.5 \cdot 0.750758 \cdot 410.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$$

$$= 123.1\ kN$$

Controlling bearing resistance in horizontal direction, $F_{b,hor,Rd} = F_{b,Rd,e}$
$$= 123.1\ kN$$

Total number of bolts, $n = n_1 \cdot n_2$
$$= 4 \cdot 1$$
$$= 4$$

$$\alpha = 0$$

Bolt row spacing, $p_1 = 70.0\ mm$

$$\beta = \frac{6 \cdot z}{n_1 \cdot (n_1 + 1) \cdot p_1}$$
$$= \frac{6 \cdot 50.4}{4 \cdot (4 + 1) \cdot 70.0}$$
$$= 0.216214$$

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

$$= \frac{4}{\sqrt{\left(\frac{(1 + 0 \cdot 4)}{99.4}\right)^2 + \left(\frac{0.216214 \cdot 4}{123.1}\right)^2}}$$
$$= 326.0\ kN$$

Applied member shear, $V_{Ed} = 200.0\ kN$

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$
$$= \frac{200.0}{326.0}$$
$$= 0.613497$$

Supported beam-bearing on fin plate (20). Reference Pg 116 (continued)

$326.0\ kN \geq 200.0\ kN$ **(OK)**

$0.613 \leq 1$ **(OK)**

Supporting member-welds (112). Reference Pg 127

Fin plate thickness, $t_p = 10.0\ mm$

FS Weld leg size, $s_{fs} = 8.0\ mm$

NS Weld leg size, $s_{ns} = 8.0\ mm$

Total effective weld throat, $a_T = 0.707 \cdot (s_{ns} + s_{fs})$

$= 0.707 \cdot (8.0 + 8.0)$

$= 11.3\ mm$

Minimum specified weld throat thickness, $a_{min} = 0.5 \cdot t_p$

$= 0.5 \cdot 10.0$

$= 5.0\ mm$

$\left(\frac{a_T}{2} = \frac{11.3}{2} = 5.7\ mm\right) \geq (a_{min} = 5.0\ mm)$

Weld is sized to develop the full strength of the fin plate.

Supporting member-local shear (36). Reference Pg 128, 129

Connection depth, $h_p = 290.0\ mm$

Supporting member thickness, $t_2 = 13.1\ mm$

Web axial load, horizontal component, $N_{w,h,Ed} = 0.0\ kN$

Applied member shear, $V_{Ed} = 200.0\ kN$

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

Shear area, $A_v = 2 \cdot h_p \cdot 0.5 \cdot t_2$

$= 2 \cdot 290.0 \cdot 0.5 \cdot 13.1$

$= 3799.0\ mm^2$

$\gamma_{M0} = 1$

Gross shear capacity of support, $R_v = \frac{\left(\frac{f_{y,2} \cdot A_v}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$

$= \frac{\left(\frac{275.0 \cdot 3799.0}{\sqrt{3} \cdot 1}\right)}{1000.0}$

$= 603.2\ kN$

$Unity = \frac{V_{Ed}}{R_v}$

$= \frac{200.0}{603.2}$

$= 0.331581$

$Shear\ capacity = R_v$

Supporting member-local shear (36). Reference Pg 128, 129 (continued)

$= 603.2\ kN$

$603.2\ kN \geq 200.0\ kN$ **(OK)**

$0.332 \leq 1$ **(OK)**

Supported beam-resistance at notch (382). Reference Pg 124

Gap, $g_h = 10.5\ mm$

Yield stress, $f_y = 275.0\ MPa$

Bottom cope length, $l_{nb} = 0.0\ mm$

Top cope length, $l_{nt} = 110.0\ mm$

Top cope depth, $d_{nt} = 50.0\ mm$

Flange thickness, $t_f = 16.0\ mm$

Flange width, $b = 179.5\ mm$

Web thickness, $t_w = 9.5\ mm$

Depth, $h_b = 412.8\ mm$

Applied member shear, $V_{Ed} = 200.0\ kN$

Supported beam-gross web shear (5). Reference Pg 121

Root radius, $r = 10.2\ mm$

Area of coped section, $A_{Tee} = (h_b - t_f - d_{nt}) \cdot t_w + b \cdot t_f + 0.429204 \cdot r^2$

$= (412.8 - 16.0 - 50.0) \cdot 9.5 + 179.5 \cdot 16.0 + 0.429204 \cdot 10.2^2$

$= 6211.3\ mm^2$

Gross area, $A = A_{Tee} - b \cdot t_f + (t_w + 2 \cdot r) \cdot \left(\frac{t_f}{2}\right)$

$= 6211.3 - 179.5 \cdot 16.0 + (9.5 + 2 \cdot 10.2) \cdot \left(\frac{16.0}{2}\right)$

$= 3578.5\ mm^2$

$\gamma_{M0} = 1$

Applied member shear, $V_{Ed} = 200.0\ kN$

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

$$= \left(\frac{200.0}{\left(\frac{275.0 \cdot 3578.5}{\sqrt{3} \cdot 1}\right)}\right) \cdot 1000.0$$

$= 0.351989$

Supported beam-resistance at notch (382). Reference Pg 124 (continued)

Shear resistance of the single coped beam, $V_{pl,N,Rd} = \dfrac{\left(\dfrac{f_y \cdot A}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$

$$= \dfrac{\left(\dfrac{275.0 \cdot 3578.5}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$$= 568.2\ kN$$

Elastic section modulus of the coped cross section, $W_{el,N} = 324070.5\ mm^3$

$\gamma_{M0} = 1$

$\left(V_{Ed} = 200.0\ kN\right) \leq \left(0.5 \cdot V_{pl,N,Rd} = 0.5 \cdot 568.2 = 284.1\ kN\right)$

Low shear condition:

Moment resistance of the coped beam, $M_{c,Rd} = \dfrac{\left(\dfrac{f_y \cdot W_{el,N}}{\gamma_{M0}}\right)}{1000000.0}$

$$= \dfrac{\left(\dfrac{275.0 \cdot 324070.5}{1}\right)}{1000000.0}$$

$$= 89.1\ kN \cdot m$$

$M_{v,N,Rd} = M_{c,Rd}$

$$= 89.1\ kN \cdot m$$

$$Shear\ capacity = \left(\frac{M_{v,N,Rd}}{\left(g_h + max\ \left(l_{nt}, l_{nb}\right)\right)}\right) \cdot 1000.0$$

$$= \left(\frac{89.1}{\left(10.5 + max\ \left(110.0, 0.0\right)\right)}\right) \cdot 1000.0$$

$$= 739.9\ kN$$

Applied member shear, $V_{Ed} = 200.0\ kN$

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

$$= \frac{200.0}{739.9}$$

$$= 0.270307$$

$739.9\ kN \geq 200.0\ kN$ **(OK)**

$0.270 \leq 1$ **(OK)**

Beam B_5 [5]

Design method

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

Overview

Section size:	533x210x92UB
Sequence:	1
ABM:	N/Assign
Plan length:	5000
Camber:	0.00 mm
Span length:	5000
Slope:	0.00 °
Material length:	4983
Plan rotation:	0.00 °

Section properties

Material grade:	S275b
Yield stress, f_y:	275.0 MPa
Tensile strength, f_u:	410.0 MPa
Depth, h:	533.1 mm
Web thickness, t_w:	10.1 mm
Flange width, b:	209.3 mm
Flange thickness, t_f:	15.6 mm
Root radius, r:	12.7 mm
Distance between web toes of fillets, d:	476.5 mm
Moment of inertia about the major axis, I_y:	552.3 10^6 mm^4

Design summary

Left end

Connection:	Shear tab
	Plate, Size as required
	No Stiffener Opposite
	Shear plate on NS, Skew holes in beam
	Combine shear plates: No
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	0
Minus Dim:	17.0 mm (AUTO)
Mtrl Setback:	17.0 mm (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	350.0 kN
Moment:	0.0 kN·m (AUTO)
Tension:	0.0 kN
Compress:	0.0 kN
Tying:	0.0 kN (AUTO)

B_5 [5] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Beam:	B_4 [7]
Section size:	610x229x140UB
End 0 elevation:	0 mm
End 1 elevation:	0 mm
Support intersection elevation:	0
Supporting beam rotation:	0.00 degrees
	(looking toward left end)
Material grade:	S275-7
Distance between web toes of fillets, d:	547.6 mm
Supporting member thickness, t_2:	13.1 mm

Node 1 notes

- A member frames to the opposite side of this member

Factored loads

Shear: 350.0 kN

Design load notes

- Reaction has been input
- Design reaction is 33.7 % of the allowable uniform steel beam load.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	S275
	Tensile strength, f_u:	410.0 MPa
	Yield stress, f_y:	275.0 MPa
	Thickness, t:	10.0 mm
	Width, b:	160.0 mm
	Depth, h:	360.0 mm
	Weld line to bolt group c.g., a:	80.5 mm
Weld:	**Weld type:**	Double fillet
	Weld leg size, s:	8.0 mm
	Total effective weld throat, a_e:	11.31 mm
	Weld metal strength, F_{exx}:	0.0 MPa
Bolts:	**Bolt type:**	8.8, Category A
	Hole type in connection:	Standard round
	Bolt diameter, d:	20
	Bolt rows, n_1:	5
	Bolt row spacing, p_1:	70.0 mm
	Bolt columns, n_2:	2
	Bolt column spacing, p_2:	60.0 mm
	Web end distance, $e_{1,w}$:	40.0 mm
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit:	3
(at dd) Not Locked:	276

Cope information

Top cope depth, d_{nt}:	50.0 mm
Top cope length, l_{nt}:	110.0 mm

Cope notes

- Cope length dimension is from the end of the beam web.
- At coped section : S_{net} = 597565.19 mm^3, h_o = 483.10 mm
- $e_{2,b}$ = 40.00 mm, $e_{1,b}$ = 40.00 mm

Expanded design calculation

Supported beam-gross web shear (5). Reference Pg 121

Yield stress, $f_y = 275.0\ MPa$

Top cope depth, $d_{nt} = 50.0\ mm$

Flange thickness, $t_f = 15.6\ mm$

Flange width, $b = 209.3\ mm$

Web thickness, $t_w = 10.1\ mm$

Full section depth, $h = 533.1\ mm$

Root radius, $r = 12.7\ mm$

Area of coped section, $A_{Tee} = \left(h - t_f - d_{nt}\right) \cdot t_w + b \cdot t_f + 0.429204 \cdot r^2$

$= (533.1 - 15.6 - 50.0) \cdot 10.1 + 209.3 \cdot 15.6 + 0.429204 \cdot 12.7^2$

$= 8056.1\ mm^2$

Gross area, $A = A_{Tee} - b \cdot t_f + (t_w + 2 \cdot r) \cdot \left(\frac{t_f}{2}\right)$

$= 8056.1 - 209.3 \cdot 15.6 + (10.1 + 2 \cdot 12.7) \cdot \left(\frac{15.6}{2}\right)$

$= 5067.9\ mm^2$

$\gamma_{M0} = 1$

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

$$= \frac{\left(\frac{275.0 \cdot 5067.9}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$= 804.6\ kN$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

$= \frac{350.0}{804.6}$

$= 0.434999$

$804.6\ kN \geq 350.0\ kN$ **(OK)**

$0.435 \leq 1$ **(OK)**

Supported beam-net web shear (4). Reference Pg 121

Tensile strength, $f_u = 410.0\ MPa$

Top cope depth, $d_{nt} = 50.0\ mm$

Bolt rows, $n_1 = 5$

Flange thickness, $t_f = 15.6\ mm$

Flange width, $b = 209.3\ mm$

Supported beam-net web shear (4). Reference Pg 121 (continued)

Web thickness, $t_w = 10.1\ mm$

Full section depth, $h = 533.1\ mm$

Root radius, $r = 12.7\ mm$

Area of coped section, $A_{Tee} = \left(h - t_f - d_{nt}\right) \cdot t_w + b \cdot t_f + 0.429204 \cdot r^2$

$= (533.1 - 15.6 - 50.0) \cdot 10.1 + 209.3 \cdot 15.6 + 0.429204 \cdot 12.7^2$

$= 8056.1\ mm^2$

Gross shear area, $A_{v,w} = A_{Tee} - b \cdot t_f + (t_w + 2 \cdot r) \cdot \left(\frac{t_f}{2}\right)$

$= 8056.1 - 209.3 \cdot 15.6 + (10.1 + 2 \cdot 12.7) \cdot \left(\frac{15.6}{2}\right)$

$= 5067.9\ mm^2$

Hole diameter, $d_0 = 22.0\ mm$

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

$= 5067.9 - 5 \cdot 22.0 \cdot 10.1$

$= 3956.9\ mm^2$

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

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

$$= \frac{\left(\frac{410.0 \cdot 3956.9}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$$

$= 851.5\ kN$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

$= \frac{350.0}{851.5}$

$= 0.41104$

$851.5\ kN \geq 350.0\ kN$ **(OK)**

$0.411 \leq 1$ **(OK)**

Supported beam-web block shear (6). Reference Pg 121

Plate thickness, $Web\ thickness = 10.1\ mm$

Yield stress, $f_y = 275.0\ MPa$

Tensile strength, $f_u = 410.0\ MPa$

Bolt column spacing, $p_2 = 60.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt rows, $n_1 = 5$

Supported beam-web block shear (6). Reference Pg 121 (continued)

Column edge distance, $e_2 = 40.0\ mm$

Row edge distance, $e_1 = 40.0\ mm$

Bolt columns, $n_2 = 2$

Hole diameter, $d_0 = 22.0\ mm$

Hole length, $l_h = 22.0\ mm$

Total length of bolt group, $p_{1,total} = 280.0\ mm$

Gross shear area, $A_{gv} = Web\ thickness \cdot (p_{1,total} + e_1)$

$= 10.1 \cdot (280.0 + 40.0)$

$= 3232.0\ mm^2$

Net shear area, $A_{nv} = Web\ thickness \cdot (p_{1,total} + e_1) - Web\ thickness \cdot (n_1 - 0.5) \cdot d_0$

$= 10.1 \cdot (280.0 + 40.0) - 10.1 \cdot (5 - 0.5) \cdot 22.0$

$= 2232.1\ mm^2$

Gross tensile area, $A_{gt} = Web\ thickness \cdot (p_2 \cdot (n_2 - 1) + e_2)$

$= 10.1 \cdot (60.0 \cdot (2 - 1) + 40.0)$

$= 1010.0\ mm^2$

Net tensile area, $A_{net} = Web\ thickness \cdot (p_2 \cdot (n_2 - 1) + e_2) - Web\ thickness \cdot (n_2 - 0.5) \cdot l_h$

$= 10.1 \cdot (60.0 \cdot (2 - 1) + 40.0) - 10.1 \cdot (2 - 0.5) \cdot 22.0$

$= 676.7\ mm^2$

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

$\gamma_{M0} = 1$

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

$$= \frac{\left(0.5 \cdot \left(\frac{410.0 \cdot 676.7}{1.1}\right) + \frac{275.0 \cdot 2232.1}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$= 480.5\ kN$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

$= \frac{350.0}{480.5}$

$= 0.728408$

$480.5\ kN \geq 350.0\ kN$ **(OK)**

$0.728 \leq 1$ **(OK)**

Supported beam-stability at notch (41). Reference Pg 125

Bottom cope length, $l_{nb} = 0.0\ mm$

Bottom cope depth, $d_{nb} = 0.0\ mm$

Top cope length, $l_{nt} = 110.0\ mm$

Top cope depth, $d_{nt} = 50.0\ mm$

Beam depth, $h_b = 533.1\ mm$

Effective top notch depth, $d_{nt,e} = max\ (d_{nt}, 0.0)$

$= max\ (50.0, 0.0)$

$= 50.0\ mm$

Effective top notch length, $l_{nt,e} = l_{nt}$

$= 110.0\ mm$

Effective bottom notch depth, $d_{nb,e} = max\ (d_{nb}, 0.0)$

$= max\ (0.0, 0.0)$

$= 0.0\ mm$

Effective bottom notch length, $l_{nb,e} = l_{nb}$

$= 0.0\ mm$

Top flange is coped.

Reduced beam depth, $h_o = h_b - d_{nt,e} - d_{nb,e}$

$= 533.1 - 50.0 - 0.0$

$= 483.1\ mm$

$(d_{nt,e} = 50.0\ mm) \leq \left(\frac{h_b}{2} = \frac{533.1}{2} = 266.6\ mm\right)$

$\left(\frac{h_b}{t_w} = \frac{533.1}{10.1} = 52.7822\right) \leq 54.3$

$(l_{nt,e} = 110.0\ mm) \leq (h_b = 533.1\ mm)$

Basic requirements are met. Local stability does not need to be checked.

Supported beam-bearing on web (20). Reference Pg 117

Column edge distance, $e_2 = 40.0\ mm$

Top vertical edge distance, $e_{1,t} = 40.0\ mm$

Connection thickness, $t = 10.1\ mm$

Connection tensile strength, $f_u = 410.0\ MPa$

Bolt column spacing, $p_2 = 60.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt columns, $n_2 = 2$

Bolt rows, $n_1 = 5$

Distance from support to center of bolt group, $z = 80.5\ mm$

Bolt diameter, $d = 20.0\ mm$

Calculate bearing resistance in vertical direction

Total length of bolt group, $p_{1,total} = 280.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt row spacing, $p_1 = 70.0\ mm$

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

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$$
$$= min\left(0.810606, \frac{800.0}{410.0}, 1\right)$$
$$= 0.810606$$

Bolt column spacing, $p_2 = 60.0\ mm$

Bolt column spacing, $p_2 = 60.0\ mm$

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, \frac{1.4 \cdot p_2}{d_0} - 1.7, 2.5\right)$$
$$= min\left(\frac{2.8 \cdot 40.0}{22.0} - 1.7, \frac{1.4 \cdot 60.0}{22.0} - 1.7, 2.5\right)$$
$$= 2.11818$$

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

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

$$= \frac{\left(\frac{2.11818 \cdot 0.810606 \cdot 410.0 \cdot 20.0 \cdot 10.1}{1.25}\right)}{1000.0}$$

$$= 113.8\ kN$$

End bolt resistance

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

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$$
$$= min\left(0.606061, \frac{800.0}{410.0}, 1\right)$$
$$= 0.606061$$

End bolt resistance (continued)

Bolt column spacing, $p_2 = 60.0\ mm$

Bolt column spacing, $p_2 = 60.0\ mm$

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, \frac{1.4 \cdot p_2}{d_0} - 1.7, 2.5\right)$$

$$= min\left(\frac{2.8 \cdot 40.0}{22.0} - 1.7, \frac{1.4 \cdot 60.0}{22.0} - 1.7, 2.5\right)$$

$$= 2.11818$$

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

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

$$= \frac{\left(\dfrac{2.11818 \cdot 0.606061 \cdot 410.0 \cdot 20.0 \cdot 10.1}{1.25}\right)}{1000.0}$$

$$= 85.1\ kN$$

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

$$= min\left(85.1, 113.8\right)$$

$$= 85.1\ kN$$

Calculate bearing resistance in horizontal direction

Controlling row edge distance in the vertical direction, $e_1 = e_{1,t}$

$$= 40.0\ mm$$

Total length of bolt group, $p_{1,total} = 280.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt column spacing, $p_2 = 60.0\ mm$

$$\alpha_d = \frac{p_2}{3 \cdot d_0} - 0.25$$

$$= \frac{60.0}{3 \cdot 22.0} - 0.25$$

$$= 0.659091$$

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

$$= min\left(0.659091, \frac{800.0}{410.0}, 1\right)$$

$$= 0.659091$$

Bolt row spacing, $p_1 = 70.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

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

$$= \frac{\left(\dfrac{2.5 \cdot 0.659091 \cdot 410.0 \cdot 20.0 \cdot 10.1}{1.25}\right)}{1000.0}$$

$$= 109.2\ kN$$

End bolt resistance

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

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$$
$$= min\left(0.606061, \frac{800.0}{410.0}, 1\right)$$
$$= 0.606061$$

Bolt row spacing, $p_1 = 70.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

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

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

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

$$= \frac{\left(\dfrac{2.5 \cdot 0.606061 \cdot 410.0 \cdot 20.0 \cdot 10.1}{1.25}\right)}{1000.0}$$

$$= 100.4\ kN$$

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

Calculate bearing resistance in horizontal direction (continued)

$= min\ (100.4, 109.2)$

$= 100.4\ kN$

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

$= 5 \cdot 2$

$= 10$

Bolt row spacing, $p_1 = 70.0\ mm$

$$l = \left(\frac{n_1}{2}\right) \cdot p_2^2 + \left(\frac{1}{6}\right) \cdot n_1 \cdot \left(n_1^2 - 1\right) \cdot p_1^2$$

$$= \left(\frac{5}{2}\right) \cdot 60.0^2 + \left(\frac{1}{6}\right) \cdot 5 \cdot \left(5^2 - 1\right) \cdot 70.0^2$$

$= 107000.0\ mm^2$

$$\alpha = \frac{z \cdot p_2}{2 \cdot l}$$

$$= \frac{80.5 \cdot 60.0}{2 \cdot 107000.0}$$

$= 0.0225561$

Bolt row spacing, $p_1 = 70.0\ mm$

$$\beta = \left(\frac{z \cdot p_1}{2 \cdot l}\right) \cdot \left(n_1 - 1\right)$$

$$= \left(\frac{80.5 \cdot 70.0}{2 \cdot 107000.0}\right) \cdot (5 - 1)$$

$= 0.105262$

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

$$= \frac{10}{\sqrt{\left(\frac{(1 + 0.0225561 \cdot 10)}{85.1}\right)^2 + \left(\frac{0.105262 \cdot 10}{100.4}\right)^2}}$$

$= 561.2\ kN$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

$$= \frac{350.0}{561.2}$$

$= 0.623664$

$561.2\ kN \geq 350.0\ kN$ **(OK)**

$0.624 \leq 1$ **(OK)**

Supported beam-fin plate block shear (6). Reference Pg 118

Plate thickness, $t_p = 10.0\ mm$

Yield stress, $f_y = 275.0\ MPa$

Tensile strength, $f_u = 410.0\ MPa$

Bolt column spacing, $p_2 = 60.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt rows, $n_1 = 5$

Column edge distance, $e_2 = 49.5\ mm$

Row edge distance, $e_1 = 40.0\ mm$

Bolt columns, $n_2 = 2$

Hole diameter, $d_0 = 22.0\ mm$

Hole length, $l_h = 22.0\ mm$

Total length of bolt group, $p_{1,total} = 280.0\ mm$

Gross shear area, $A_{gv} = t_p \cdot (p_{1,total} + e_1)$

$= 10.0 \cdot (280.0 + 40.0)$

$= 3200.0\ mm^2$

Net shear area, $A_{nv} = t_p \cdot (p_{1,total} + e_1) - t_p \cdot (n_1 - 0.5) \cdot d_0$

$= 10.0 \cdot (280.0 + 40.0) - 10.0 \cdot (5 - 0.5) \cdot 22.0$

$= 2210.0\ mm^2$

Gross tensile area, $A_{gt} = t_p \cdot (p_2 \cdot (n_2 - 1) + e_2)$

$= 10.0 \cdot (60.0 \cdot (2 - 1) + 49.5)$

$= 1095.5\ mm^2$

Net tensile area, $A_{net} = t_p \cdot (p_2 \cdot (n_2 - 1) + e_2) - t_p \cdot (n_2 - 0.5) \cdot l_h$

$= 10.0 \cdot (60.0 \cdot (2 - 1) + 49.5) - 10.0 \cdot (2 - 0.5) \cdot 22.0$

$= 765.5\ mm^2$

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

$\gamma_{M0} = 1$

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

$$= \frac{\left(0.5 \cdot \left(\frac{410.0 \cdot 765.5}{1.1}\right) + \frac{275.0 \cdot 2210.0}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$= 493.5\ kN$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

$$= \frac{350.0}{493.5}$$

Supported beam-fin plate block shear (6). Reference Pg 118 (continued)

$= 0.70922$

$493.5\ kN \geq 350.0\ kN$ **(OK)**

$0.709 \leq 1$ **(OK)**

Supported beam-fin plate gross shear (38). Reference Pg 118

Depth, $h = 360.0\ mm$

Plate thickness, $t_p = 10.0\ mm$

Plate yield stress, $f_{y,p} = 275.0\ MPa$

Applied member shear, $V_{Ed} = 350.0\ kN$

Tension force, horizontal component, $N_{h,t,Ed} = 0.0\ kN$

Compression force, horizontal component, $N_{h,c,Ed} = 0.0\ kN$

Gross area, $A = h \cdot t_p$

$= 360.0 \cdot 10.0$

$= 3600.0\ mm^2$

$\gamma_{M0} = 1$

$$\text{Plate shear resistance, } V_{Rd} = \frac{\left(\frac{\left(\frac{f_{y,p} \cdot A}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1.27}\right)}{1000.0}$$

$$= \frac{\left(\frac{\left(\frac{275.0 \cdot 3600.0}{\sqrt{3} \cdot 1}\right)}{1.27}\right)}{1000.0}$$

$= 450.1\ kN$

$Unity = \frac{V_{Ed}}{V_{Rd}}$

$= \frac{350.0}{450.1}$

$= 0.777674$

$Shear\ capacity = V_{Rd}$

$= 450.1\ kN$

$450.1\ kN \geq 350.0\ kN$ **(OK)**

$0.778 \leq 1$ **(OK)**

Connection-net shear rupture (21). Reference Pg 118

Connection tensile strength, $f_u = 410.0\ MPa$

Bolt rows, $n_1 = 5$

Plate thickness, $t_p = 10.0\ mm$

Plate depth, $h_p = 360.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

Net shear area, $A_{nv} = t_p \cdot (h_p - n_1 \cdot d_0)$

$= 10.0 \cdot (360.0 - 5 \cdot 22.0)$

$= 2500.0\ mm^2$

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

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

$= \dfrac{\left(\dfrac{410.0 \cdot 2500.0}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$

$= 538.0\ kN$

$Shear\ capacity = V_{Rd}$

$= 538.0\ kN$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

$= \dfrac{350.0}{538.0}$

$= 0.650558$

$538.0\ kN \geq 350.0\ kN$ **(OK)**

$0.651 \leq 1$ **(OK)**

Supported beam-fin plate bending and lateral torsional buckling (19). Reference Pg 119

Plate yield stress, $f_{y,p} = 275.0\ MPa$

Plate thickness, $t_p = 10.0\ mm$

Plate depth, $h_p = 360.0\ mm$

Distance from weld to centroid of bolt group, $z = 80.5\ mm$

Bolt column spacing, $p_2 = 60.0\ mm$

Bolt columns, $n_2 = 2$

Elastic section modulus, $W_{el,p} = \dfrac{t_p \cdot h_p^2}{6}$

$= \dfrac{10.0 \cdot 360.0^2}{6}$

$= 216000.0\ mm^3$

Supported beam-fin plate bending and lateral torsional buckling (19). Reference Pg 119 (continued)

$\gamma_{M0} = 1$

Distance from weld to first column of bolts, $z_p = z - \frac{p_2 \cdot (n_2 - 1)}{2}$

$= 80.5 - \frac{60.0 \cdot (2 - 1)}{2}$

$= 50.4\ mm$

$(z_p = 50.4\ mm) \leq \left(\frac{t_p}{0.15} = \frac{10.0}{0.15} = 66.7\ mm\right)$

Short Fin Plate

$$Shear\ capacity = \frac{\left(\frac{\left(\frac{W_{el,p}}{z}\right) \cdot f_{y,p}}{\gamma_{M0}}\right)}{1000.0}$$

$$= \frac{\left(\frac{\left(\frac{216000.0}{80.5}\right) \cdot 275.0}{1}\right)}{1000.0}$$

$= 738.3\ kN$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

$= \frac{350.0}{738.3}$

$= 0.474062$

$738.3\ kN \geq 350.0\ kN$ **(OK)**

$0.474 \leq 1$ **(OK)**

Supported beam-bolt shear (3). Reference Pg 115

Number of shear planes, $N_s = 1$

Bolt column spacing, $p_2 = 60.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt columns, $n_2 = 2$

Bolt rows, $n_1 = 5$

Distance from weld to center of bolt group, $z = 80.5\ mm$

Supported beam-bolt shear (3). Reference Pg 115 (continued)

Bolt row spacing, $p_1 = 70.0\ mm$

$$\alpha = \frac{z \cdot p_2}{2 \cdot \left(\left(\frac{n_1}{2}\right) \cdot p_2^2 + \left(\frac{1}{6}\right) \cdot n_1 \cdot \left(n_1^2 - 1\right) \cdot p_1^2\right)}$$

$$= \frac{80.5 \cdot 60.0}{2 \cdot \left(\left(\frac{5}{2}\right) \cdot 60.0^2 + \left(\frac{1}{6}\right) \cdot 5 \cdot \left(5^2 - 1\right) \cdot 70.0^2\right)}$$

$$= 0.0225561$$

Total length of bolt group, $p_{1,total} = 280.0\ mm$

$$\beta = \frac{z \cdot p_{1,total}}{2 \cdot \left(\left(\frac{n_1}{2}\right) \cdot p_2^2 + \left(\frac{1}{6}\right) \cdot n_1 \cdot \left(n_1^2 - 1\right) \cdot p_1^2\right)}$$

$$= \frac{80.5 \cdot 280.0}{2 \cdot \left(\left(\frac{5}{2}\right) \cdot 60.0^2 + \left(\frac{1}{6}\right) \cdot 5 \cdot \left(5^2 - 1\right) \cdot 70.0^2\right)}$$

$$= 0.105262$$

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

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

$$= 94.0\ kN$$

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

$$= \left(\frac{5 \cdot 2}{\sqrt{\left(1 + 0.0225561 \cdot 5 \cdot 2\right)^2 + \left(0.105262 \cdot 5 \cdot 2\right)^2}}\right) \cdot 94.0$$

$$= 581.9\ kN$$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

$$= \frac{350.0}{581.9}$$

$$= 0.601478$$

Supported beam-bolt shear (3). Reference Pg 115 (continued)

$581.9\ kN \geq 350.0\ kN$ **(OK)**

$0.601 \leq 1$ **(OK)**

Supported beam-bearing on fin plate (20). Reference Pg 116

Column edge distance, $e_2 = 49.5\ mm$

Bottom vertical edge distance, $e_{1,b} = 40.0\ mm$

Top vertical edge distance, $e_{1,t} = 40.0\ mm$

Connection thickness, $t = 10.0\ mm$

Connection tensile strength, $f_u = 410.0\ MPa$

Bolt column spacing, $p_2 = 60.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt columns, $n_2 = 2$

Bolt rows, $n_1 = 5$

Distance from support to center of bolt group, $z = 80.5\ mm$

Bolt diameter, $d = 20.0\ mm$

Calculate bearing resistance in vertical direction

Total length of bolt group, $p_{1,total} = 280.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt row spacing, $p_1 = 70.0\ mm$

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

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right) = min\left(0.810606, \frac{800.0}{410.0}, 1\right) = 0.810606$$

Bolt column spacing, $p_2 = 60.0\ mm$

Bolt column spacing, $p_2 = 60.0\ mm$

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, \frac{1.4 \cdot p_2}{d_0} - 1.7, 2.5\right) = min\left(\frac{2.8 \cdot 49.5}{22.0} - 1.7, \frac{1.4 \cdot 60.0}{22.0} - 1.7, 2.5\right) = 2.11818$$

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

Inner bolt resistance (continued)

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

$= \dfrac{\left(\dfrac{2.11818 \cdot 0.810606 \cdot 410.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$

$= 112.6\ kN$

End bolt resistance

$\alpha_d = \dfrac{e_{1,t}}{3 \cdot d_0}$

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

$= 0.606061$

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

$= min\left(0.606061, \dfrac{800.0}{410.0}, 1\right)$

$= 0.606061$

Bolt column spacing, $p_2 = 60.0\ mm$

Bolt column spacing, $p_2 = 60.0\ mm$

$k_1 = min\left(\dfrac{2.8 \cdot e_2}{d_0} - 1.7, \dfrac{1.4 \cdot p_2}{d_0} - 1.7, 2.5\right)$

$= min\left(\dfrac{2.8 \cdot 49.5}{22.0} - 1.7, \dfrac{1.4 \cdot 60.0}{22.0} - 1.7, 2.5\right)$

$= 2.11818$

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

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

$= \dfrac{\left(\dfrac{2.11818 \cdot 0.606061 \cdot 410.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$

$= 84.2\ kN$

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

$= min\ (84.2, 112.6)$

$= 84.2\ kN$

Calculate bearing resistance in horizontal direction

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

$= min\ (40.0, 40.0)$

Calculate bearing resistance in horizontal direction (continued)

$= 40.0\ mm$

Total length of bolt group, $p_{1,total} = 280.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt column spacing, $p_2 = 60.0\ mm$

$$\alpha_d = \frac{p_2}{3 \cdot d_0} - 0.25$$
$$= \frac{60.0}{3 \cdot 22.0} - 0.25$$
$$= 0.659091$$

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$$
$$= min\left(0.659091, \frac{800.0}{410.0}, 1\right)$$
$$= 0.659091$$

Bolt row spacing, $p_1 = 70.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

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

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

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

$$= \frac{\left(\dfrac{2.5 \cdot 0.659091 \cdot 410.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$$

$$= 108.1\ kN$$

End bolt resistance

$$\alpha_d = \frac{e_2}{3 \cdot d_0}$$
$$= \frac{49.5}{3 \cdot 22.0}$$
$$= 0.750758$$

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

End bolt resistance (continued)

$= min\left(0.750758, \frac{800.0}{410.0}, 1\right)$

$= 0.750758$

Bolt row spacing, $p_1 = 70.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

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

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

$= 2.5$

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

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

$= \frac{\left(\frac{2.5 \cdot 0.750758 \cdot 410.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$

$= 123.1\ kN$

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

$= min\ (123.1, 108.1)$

$= 108.1\ kN$

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

$= 5 \cdot 2$

$= 10$

Bolt row spacing, $p_1 = 70.0\ mm$

$l = \left(\frac{n_1}{2}\right) \cdot p_2^2 + \left(\frac{1}{6}\right) \cdot n_1 \cdot \left(n_1^2 - 1\right) \cdot p_1^2$

$= \left(\frac{5}{2}\right) \cdot 60.0^2 + \left(\frac{1}{6}\right) \cdot 5 \cdot \left(5^2 - 1\right) \cdot 70.0^2$

$= 107000.0\ mm^2$

$\alpha = \frac{z \cdot p_2}{2 \cdot l}$

$= \frac{80.5 \cdot 60.0}{2 \cdot 107000.0}$

$= 0.0225561$

Bolt row spacing, $p_1 = 70.0\ mm$

$\beta = \left(\frac{z \cdot p_1}{2 \cdot l}\right) \cdot \left(n_1 - 1\right)$

$= \left(\frac{80.5 \cdot 70.0}{2 \cdot 107000.0}\right) \cdot (5 - 1)$

$= 0.105262$

Supported beam-bearing on fin plate (20). Reference Pg 116 (continued)

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

$$= \frac{10}{\sqrt{\left(\frac{(1 + 0.0225561 \cdot 10)}{84.2}\right)^2 + \left(\frac{0.105262 \cdot 10}{108.1}\right)^2}}$$

$= 571.1\ kN$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

$$= \frac{350.0}{571.1}$$

$= 0.612853$

$571.1\ kN \geq 350.0\ kN$ **(OK)**

$0.613 \leq 1$ **(OK)**

Supporting member-welds (112). Reference Pg 127

Fin plate thickness, $t_p = 10.0\ mm$

FS Weld leg size, $s_{fs} = 8.0\ mm$

NS Weld leg size, $s_{ns} = 8.0\ mm$

Total effective weld throat, $a_T = 0.707 \cdot (s_{ns} + s_{fs})$

$= 0.707 \cdot (8.0 + 8.0)$

$= 11.3\ mm$

Minimum specified weld throat thickness, $a_{min} = 0.5 \cdot t_p$

$= 0.5 \cdot 10.0$

$= 5.0\ mm$

$$\left(\frac{a_T}{2} = \frac{11.3}{2} = 5.7\ mm\right) \geq (a_{min} = 5.0\ mm)$$

Weld is sized to develop the full strength of the fin plate.

Supporting member-local shear (36). Reference Pg 128, 129

Connection depth, $h_p = 360.0\ mm$

Supporting member thickness, $t_2 = 13.1\ mm$

Web axial load, horizontal component, $N_{w,h,Ed} = 0.0\ kN$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

Shear area, $A_v = 2 \cdot h_p \cdot 0.5 \cdot t_2$

$= 2 \cdot 360.0 \cdot 0.5 \cdot 13.1$

Supporting member-local shear (36). Reference Pg 128, 129 (continued)

$= 4716.0\ mm^2$

$\gamma_{M0} = 1$

Gross shear capacity of support, $R_v = \dfrac{\left(\dfrac{f_{y,2} \cdot A_v}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{275.0 \cdot 4716.0}{\sqrt{3} \cdot 1}\right)}{1000.0}$

$= 748.8\ kN$

$Unity = \dfrac{V_{Ed}}{R_v}$

$= \dfrac{350.0}{748.8}$

$= 0.467437$

$Shear\ capacity = R_v$

$= 748.8\ kN$

$748.8\ kN \geq 350.0\ kN$ **(OK)**

$0.467 \leq 1$ **(OK)**

Supported beam-resistance at notch (382). Reference Pg 124

Gap, $g_h = 10.5\ mm$

Horizontal edge distance, $e_2 = 40.0\ mm$

Bolt column spacing, $p_2 = 60.0\ mm$

Bolt columns, $n_2 = 2$

Yield stress, $f_y = 275.0\ MPa$

Bottom flange flush cut length, $l_{flg_{cut,bot}} = 0.0\ mm$

Top flange flush cut length, $l_{flg_{cut,top}} = 0.0\ mm$

Bottom cope length, $l_{nb} = 0.0\ mm$

Top cope length, $l_{nt} = 110.0\ mm$

Top cope depth, $d_{nt} = 50.0\ mm$

Flange thickness, $t_f = 15.6\ mm$

Flange width, $b = 209.3\ mm$

Web thickness, $t_w = 10.1\ mm$

Depth, $h_b = 533.1\ mm$

Applied member shear, $V_{Ed} = 350.0\ kN$

Supported beam-gross web shear (5). Reference Pg 121

Root radius, $r = 12.7\ mm$

Area of coped section, $A_{Tee} = \left(h_b - t_f - d_{nt}\right) \cdot t_w + b \cdot t_f + 0.429204 \cdot r^2$

Supported beam-gross web shear (5). Reference Pg 121 (continued)

$$= \left(533.1 - 15.6 - 50.0\right) \cdot 10.1 + 209.3 \cdot 15.6 + 0.429204 \cdot 12.7^2$$

$$= 8056.1\ mm^2$$

Gross area, $A = A_{Tee} - b \cdot t_f + \left(t_w + 2 \cdot r\right) \cdot \left(\frac{t_f}{2}\right)$

$$= 8056.1 - 209.3 \cdot 15.6 + \left(10.1 + 2 \cdot 12.7\right) \cdot \left(\frac{15.6}{2}\right)$$

$$= 5067.9\ mm^2$$

$\gamma_{M0} = 1$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

$$= \left(\frac{350.0}{\left(\frac{275.0 \cdot 5067.9}{\sqrt{3} \cdot 1}\right)}\right) \cdot 1000.0$$

$$= 0.434999$$

Shear resistance of the single coped beam, $V_{pl,N,Rd} = \frac{\left(\frac{f_y \cdot A}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$

$$= \frac{\left(\frac{275.0 \cdot 5067.9}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$$= 804.6\ kN$$

Elastic section modulus of the coped cross section, $W_{el,N} = 597565.2\ mm^3$

$\gamma_{M0} = 1$

$\left(V_{Ed} = 350.0\ kN\right) \leq \left(0.5 \cdot V_{pl,N,Rd} = 0.5 \cdot 804.6 = 402.3\ kN\right)$

Low shear condition:

Moment resistance of the coped beam, $M_{c,Rd} = \frac{\left(\frac{f_y \cdot W_{el,N}}{\gamma_{M0}}\right)}{1000000.0}$

$$= \frac{\left(\frac{275.0 \cdot 597565.2}{1}\right)}{1000000.0}$$

$$= 164.3\ kN \cdot m$$

$M_{v,N,Rd} = M_{c,Rd}$

Supported beam-resistance at notch (382). Reference Pg 124 (continued)

$= 164.3\ kN \cdot m$

$x_N = \left| max\ (l_{nt}, l_{nb}, l_{flg\ cut,top}, l_{flg\ cut,bot}) - (e_2 + (n_2 - 1) \cdot p_2) \right|$

$= \left| max\ (110.0, 0.0, 0.0, 0.0) - (40.0 + (2 - 1) \cdot 60.0) \right|$

$= 10.0\ mm$

$$Shear\ capacity = \left(\frac{M_{v,N,Rd}}{max\ (g_h + max\ (l_{nt}, l_{nb})_h + e_2 + p_2)} \right) \cdot 1000.0$$

$$= \left(\frac{164.3}{max\ (10.5 + max\ (110.0, 0.0) + 40.0 + 60.0)} \right) \cdot 1000.0$$

$= 1364.3\ kN$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

$$= \frac{350.0}{1364.3}$$

$= 0.256542$

$1364.3\ kN \geq 350.0\ kN$ **(OK)**

$0.257 \leq 1$ **(OK)**

Supported beam-notched web shear (380). Reference Pg 122

Gap, $g_h = 10.5\ mm$

Horizontal edge distance, $e_2 = 40.0\ mm$

Bolt column spacing, $p_2 = 60.0\ mm$

Bolt rows, $n_1 = 5$

Tensile strength, $f_u = 410.0\ MPa$

Yield stress, $f_y = 275.0\ MPa$

Top cope depth, $d_{nt} = 50.0\ mm$

Flange thickness, $t_f = 15.6\ mm$

Flange width, $b = 209.3\ mm$

Web thickness, $t_w = 10.1\ mm$

Depth, $h = 533.1\ mm$

Applied member shear, $V_{Ed} = 350.0\ kN$

Supported beam-gross web shear (5). Reference Pg 121

Root radius, $r = 12.7\ mm$

Area of coped section, $A_{Tee} = (h - t_f - d_{nt}) \cdot t_w + b \cdot t_f + 0.429204 \cdot r^2$

$= (533.1 - 15.6 - 50.0) \cdot 10.1 + 209.3 \cdot 15.6 + 0.429204 \cdot 12.7^2$

$= 8056.1\ mm^2$

$$Gross\ area,\ A = A_{Tee} - b \cdot t_f + (t_w + 2 \cdot r) \cdot \left(\frac{t_f}{2} \right)$$

$$= 8056.1 - 209.3 \cdot 15.6 + (10.1 + 2 \cdot 12.7) \cdot \left(\frac{15.6}{2} \right)$$

Supported beam-gross web shear (5). Reference Pg 121 (continued)

$= 5067.9\ mm^2$

$\gamma_{M0} = 1$

Applied member shear, $V_{Ed} = 350.0\ kN$

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

$$= \left(\frac{350.0}{\left(\frac{275.0 \cdot 5067.9}{\sqrt{3} \cdot 1} \right)} \right) \cdot 1000.0$$

$= 0.434999$

Gross shear capacity, $V_{Rd,g} = \dfrac{\left(\frac{f_y \cdot A}{\sqrt{3} \cdot \gamma_{M0}} \right)}{1000.0}$

$$= \frac{\left(\frac{275.0 \cdot 5067.9}{\sqrt{3} \cdot 1} \right)}{1000.0}$$

$= 804.6\ kN$

Supported beam-net web shear (4). Reference Pg 121

Root radius, $r = 12.7\ mm$

Area of coped section, $A_{Tee} = \left(h - t_f - d_{nt}\right) \cdot t_w + b \cdot t_f + 0.429204 \cdot r^2$

$= (533.1 - 15.6 - 50.0) \cdot 10.1 + 209.3 \cdot 15.6 + 0.429204 \cdot 12.7^2$

$= 8056.1\ mm^2$

Gross shear area, $A_{v,w} = A_{Tee} - b \cdot t_f + (t_w + 2 \cdot r) \cdot \left(\frac{t_f}{2} \right)$

$= 8056.1 - 209.3 \cdot 15.6 + (10.1 + 2 \cdot 12.7) \cdot \left(\frac{15.6}{2} \right)$

$= 5067.9\ mm^2$

Hole diameter, $d_0 = 22.0\ mm$

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

$= 5067.9 - 5 \cdot 22.0 \cdot 10.1$

$= 3956.9\ mm^2$

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

Applied member shear, $V_{Ed} = 350.0\ kN$

Supported beam-net web shear (4). Reference Pg 121 (continued)

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

$$= \left(\frac{350.0}{\left(\frac{410.0 \cdot 3956.9}{\sqrt{3} \cdot 1.1} \right)} \right) \cdot 1000.0$$

$$= 0.41104$$

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

$$= \frac{\left(\frac{410.0 \cdot 3956.9}{\sqrt{3} \cdot 1.1} \right)}{1000.0}$$

$$= 851.5\ kN$$

Shear resistance of the single coped beam, $V_{pl,N,Rd} = min\ \left(V_{Rd,g}, V_{Rd,n}\right)$

$$= min\ (804.6, 851.5)$$

$$= 804.6\ kN$$

Elastic section modulus of the coped cross section, $W_{el,N} = 597565.2\ mm^3$

$\gamma_{M0} = 1$

$\left(V_{Ed} = 350.0\ kN\right) \leq \left(0.5 \cdot V_{pl,N,Rd} = 0.5 \cdot 804.6 = 402.3\ kN\right)$

Low shear condition:

Moment resistance of the coped beam, $M_{c,Rd} = \frac{\left(\frac{f_y \cdot W_{el,N}}{\gamma_{M0}} \right)}{1000000.0}$

$$= \frac{\left(\frac{275.0 \cdot 597565.2}{1} \right)}{1000000.0}$$

$$= 164.3\ kN \cdot m$$

$$Shear\ capacity = \left(\frac{M_{c,Rd}}{(g_h + e_2 + p_2)} \right) \cdot 1000.0$$

$$= \left(\frac{164.3}{(10.5 + 40.0 + 60.0)} \right) \cdot 1000.0$$

$$= 1487.8\ kN$$

Applied member shear, $V_{Ed} = 350.0\ kN$

Supported beam-notched web shear (380). Reference Pg 122 (continued)

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$
$$= \frac{350.0}{1487.8}$$
$$= 0.235247$$

$1487.8\ kN \geq 350.0\ kN$ **(OK)**

$0.235 \leq 1$ **(OK)**

Results summary

Shear Tab on right end of Beam B_3 [4]

Design calculation summary for member [4], right end

Desc. Ref.:	Calc. Num.	Unity Ratio	Resistance	Design Force	Green book ref.
2(i):	(3)	0.703	284.4 kN	200.0 kN	Pg 115
2(ii):	(20)	0.613	326.0 kN	200.0 kN	Pg 116
2(iii):	(20)	0.700	285.7 kN	200.0 kN	Pg 117
3(i):	(38)	0.552	362.5 kN	200.0 kN	Pg 118
3(ii):	(21)	0.460	434.7 kN	200.0 kN	Pg 118
3(iii):	(6)	0.577	346.5 kN	200.0 kN	Pg 118
3(iv):	(19)	0.262	764.0 kN	200.0 kN	Pg 119
4(i):	(5)	0.352	568.2 kN	200.0 kN	Pg 121
4(ii):	(4)	0.339	590.2 kN	200.0 kN	Pg 121
4(iii):	(6)	0.640	312.3 kN	200.0 kN	Pg 121
5:	(382)	0.270	739.9 kN	200.0 kN	Pg 124
10:	(36)	0.332	603.2 kN	200.0 kN	Pg 128, 129

Check number reference

2(i):	Supported beam-bolt shear
2(ii):	Supported beam-bearing on fin plate
2(iii):	Supported beam-bearing on web
3(i):	Supported beam-fin gross shear
3(ii):	Connection-net shear rupture
3(iii):	Supported beam-fin block shear
3(iv):	Supported beam-fin bending & LTB
4(i):	Supported beam-gross web shear
4(ii):	Supported beam-net web shear
4(iii):	Supported beam-web block shear
5:	Supported beam-resistance at notch
10:	Supporting member-local shear

Connection strength

	Unity ratio:
Shear:	0.703

Notes and conclusions

- Design based on Simple Joints to Eurocode 3, Chapter 5.
- t_p
- t_w
- tp/zp >= 0.15 --> short fin plate
- See 'Fin plate connection' notes for design criteria applicable to this connection. ⚠
- CONNECTION IS OK
  - Design resistance equals or exceeds design forces.
- Weld size satisfies basic requirements of Check 8 from the Green Book.

Shear Tab on left end of Beam B_5 [5]

Design calculation summary for member [5], left end

Desc. Ref.:	Calc. Num.	Unity Ratio	Resistance	Design Force	Green book ref.
2(i):	(3)	0.601	581.9 kN	350.0 kN	Pg 115
2(ii):	(20)	0.613	571.1 kN	350.0 kN	Pg 116
2(iii):	(20)	0.624	561.2 kN	350.0 kN	Pg 117
3(i):	(38)	0.778	450.1 kN	350.0 kN	Pg 118
3(ii):	(21)	0.651	538.0 kN	350.0 kN	Pg 118
3(iii):	(6)	0.709	493.5 kN	350.0 kN	Pg 118
3(iv):	(19)	0.474	738.3 kN	350.0 kN	Pg 119
4(i):	(5)	0.435	804.6 kN	350.0 kN	Pg 121
4(ii):	(4)	0.411	851.5 kN	350.0 kN	Pg 121
4(iii):	(6)	0.728	480.5 kN	350.0 kN	Pg 121
4(iv):	(380)	0.235	1487.8 kN	350.0 kN	Pg 122
5:	(382)	0.257	1364.3 kN	350.0 kN	Pg 124
10:	(36)	0.467	748.8 kN	350.0 kN	Pg 128, 129

Check number reference

2(i):	Supported beam-bolt shear
2(ii):	Supported beam-bearing on fin plate
2(iii):	Supported beam-bearing on web
3(i):	Supported beam-fin gross shear
3(ii):	Connection-net shear rupture
3(iii):	Supported beam-fin block shear
3(iv):	Supported beam-fin bending & LTB
4(i):	Supported beam-gross web shear
4(ii):	Supported beam-net web shear
4(iii):	Supported beam-web block shear
4(iv):	Supported beam-notched web shear
5:	Supported beam-resistance at notch
10:	Supporting member-local shear

Connection strength

	Unity ratio:
Shear:	0.778

Notes and conclusions

- Design based on Simple Joints to Eurocode 3, Chapter 5.
- t_p
- t_w > 0.5d
- tp/zp >= 0.15 --> short fin plate
- See 'Fin plate connection' notes for design criteria applicable to this connection. ⚠
- CONNECTION IS OK
    - Design resistance equals or exceeds design forces.
- Weld size satisfies basic requirements of Check 8 from the Green Book.

Additional notes

Design Example 1 - Fin plate, Page 137,
P358 Simple Joints.pdf