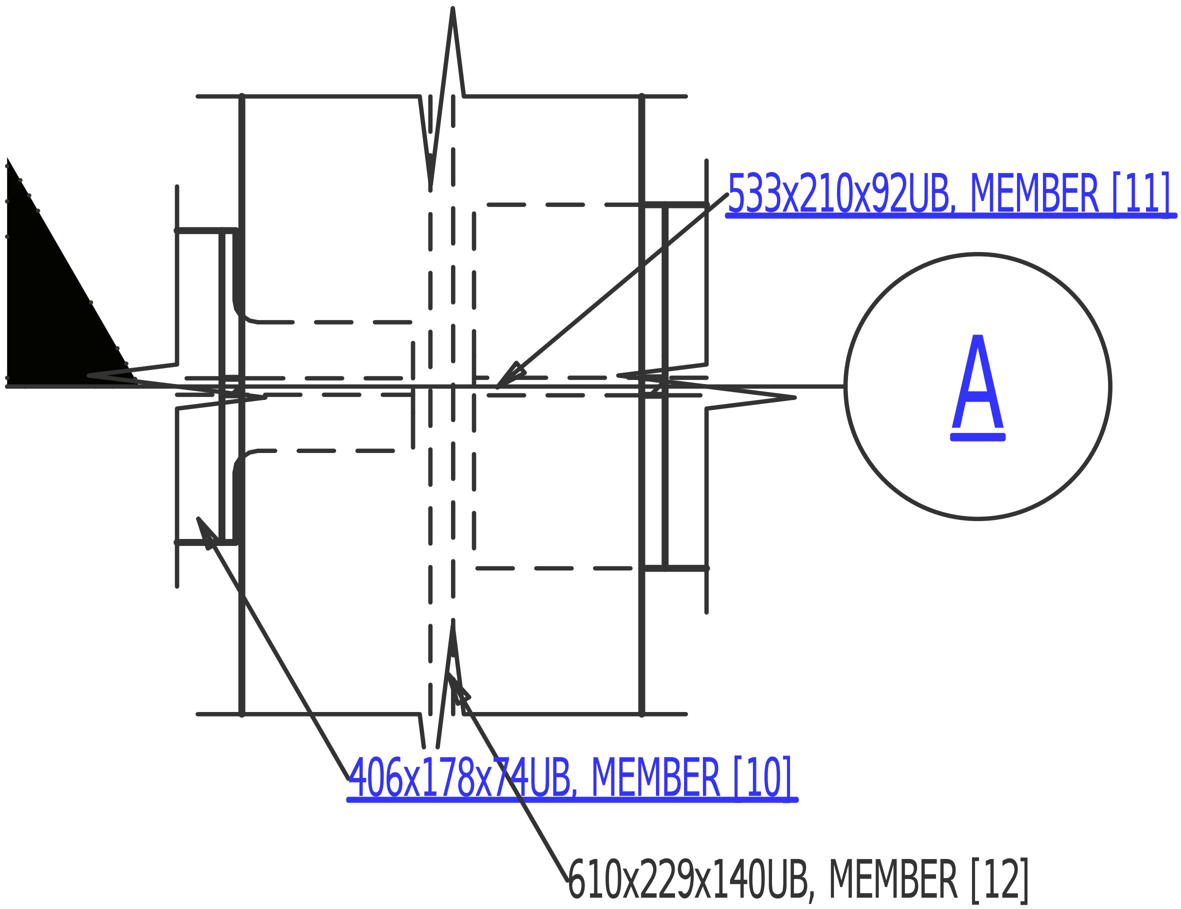
SDS2 Steel Connection Design: Connection Cube Report

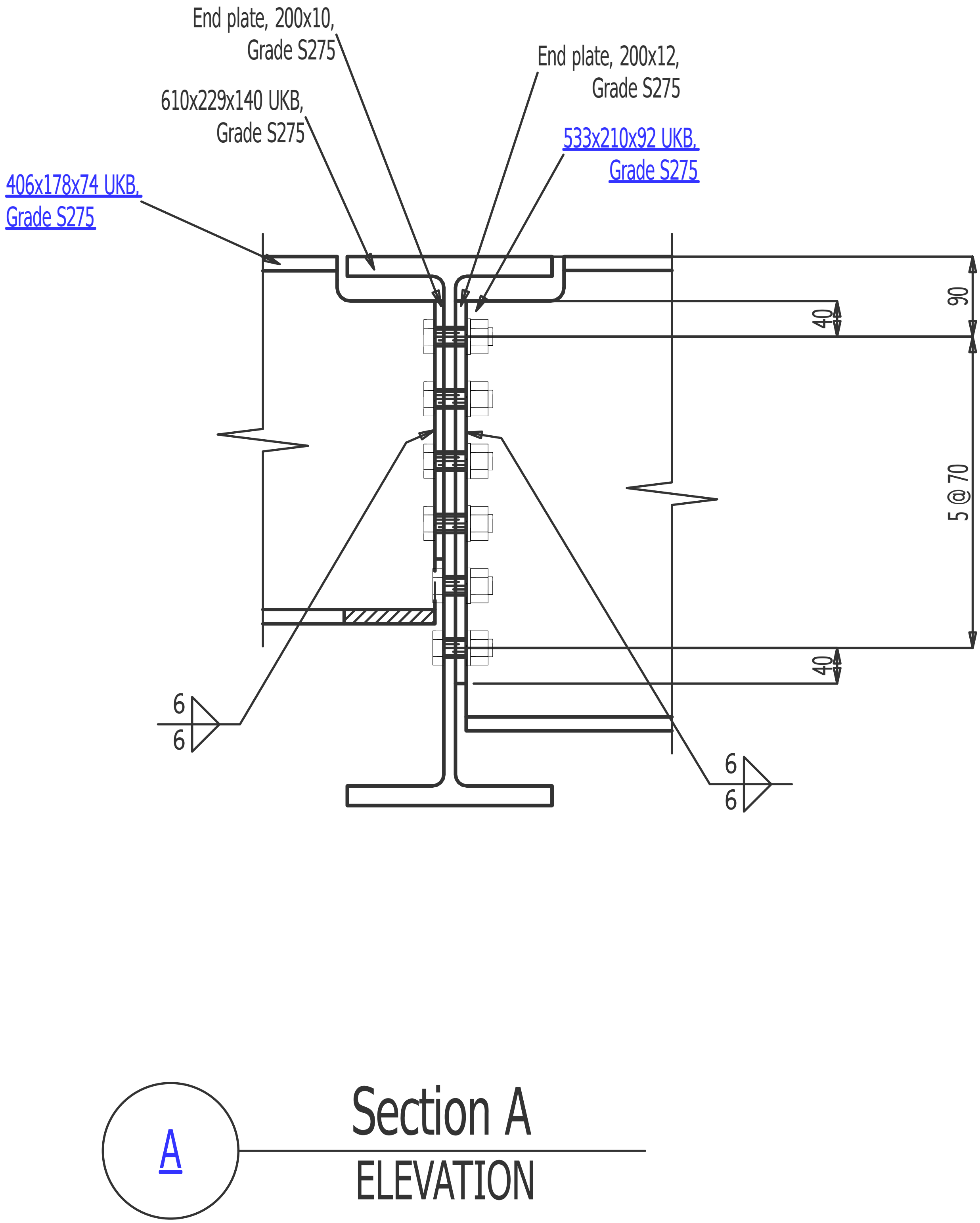
Cube: Partial depth end plate
Revision: 0
Project: 1046_Validation_Eurocode_3UK
Engineer:
Fabricator: Validation_Eurocode_3UK

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

Partial depth end plate [4] at X=5000, Y=-5000 Elev=-267



TOP SIDE VIEW


End plate, 200x10,
Grade S275
End plate, 200x12,
Grade S275
610x229x140 UKB,
Grade S275
533x210x92 UKB,
Grade S275
406x178x74 UKB,
Grade S275
90
40
5 @ 70
40
6
6
6
6
A
Section A
ELEVATION

Beam B_10 [10]

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:	S275
Yield stress, f_y:	275.0 MPa
Tensile strength, f_u:	430.0 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:	End plate
	Wide gage, Partial depth end plate
	Welded extended tee: No
	Safety Automatic
	Expand Vertical bolt spacing: Auto
	Use web extension: If Required
Elevation:	0
Minus Dim:	6.5 mm (AUTO)
Mtrl Setback:	16.5 mm (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	340.0 kN
Moment:	0.0 kN·m (AUTO)
Tension:	0.0 kN
Compress:	0.0 kN
Tying:	0.0 kN (AUTO)

B_10 [10] Connection strength check: RIGHT END

Member end summary

Connecting nodes

Node 1

Beam:	B_11 [12]
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:	S275a
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

$Shear$:	340.0 kN

Design load notes

- Reaction has been input
- Design reaction is 51.5 % of the allowable uniform steel beam load.

Connection summary

- END 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
	Depth, h:	290.0 mm
	Width, b:	200.0 mm
Welds:	**Weld type:**	Double fillet
	Weld leg size, s:	6.0 mm
	Weld metal strength, F_{exx}:	0.0 MPa
	Total effective weld throat, a_e:	8.5 mm
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 gage, p_3:	140.0 mm
	Row edge distance, e_1:	40.0 mm
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit:	2
(at dd) Not Locked:	285

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} = 278763.76 mm^3, h_o = 362.80 mm

Flange cut information

Btm flange cut width: 74.0 mm

Expanded design calculation

Supported beam-web shear (8). Reference Pg 17

Beam yield stress, $f_{y,b1} = 275.0\ MPa$

Connection depth, $h_p = 290.0\ mm$

Web thickness, $t_w = 9.5\ mm$

Shear Area, $A_g = 0.9 \cdot h_p \cdot t_w$

$= 0.9 \cdot 290.0 \cdot 9.5$

$= 2479.5\ mm^2$

$\gamma_{M0} = 1$

Allowable shear capacity, $R_v = \dfrac{\left(\dfrac{f_{y,b1} \cdot A_g}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{275.0 \cdot 2479.5}{\sqrt{3} \cdot 1}\right)}{1000.0}$

$= 393.7\ kN$

Applied member shear, $V_{Ed} = 340.0\ kN$

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

$= \dfrac{340.0}{393.7}$

$= 0.863661$

$Shear\ capacity = R_v$

$= 393.7\ kN$

$393.7\ kN \geq 340.0\ kN$ **(OK)**

$0.864 \leq 1$ **(OK)**

Supported beam-stability at notch (41). Reference Pg 19

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, $d_{nb,e} = max\ (d_{nb}, 0.0)$

Supported beam-stability at notch (41). Reference Pg 19 (continued)

$= max\ (0.0{,}0.0)$

$= 0.0\ mm$

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

$= 0.0\ mm$

Top flange is coped.

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

$= 412.8 - 50.0 - 0.0$

$= 362.8\ mm$

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

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

$(l_{nt,e} = 110.0\ mm) \le (h_b = 412.8\ mm)$

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

Connection-block tearing (252). Reference Pg 23

Yield stress, $f_y = 275.0\ MPa$

Tensile strength, $f_u = 410.0\ MPa$

Connection thickness, $t = 10.0\ mm$

Bolt column spacing, $p_2 = 0.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt columns, $n_2 = 1$

NS bolt rows, $n_{1,NS} = 4$

Horizontal edge distance, $e_2 = 30.0\ mm$

Vertical edge distance, $e_1 = 40.0\ mm$

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

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

$= 10.0 \cdot (210.0 + 40.0)$

$= 2500.0\ mm^2$

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

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

$= 1730.0\ mm^2$

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

$= 10.0 \cdot (0.0 \cdot (1 - 1) + 30.0)$

$= 300.0\ mm^2$

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

Calculate block shear capacity of each side (continued)

$= 10.0 \cdot (0.0 \cdot (1 - 1) + 30.0) - 10.0 \cdot (1 - 0.5) \cdot 22.0$

$= 190.0\ mm^2$

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

$\gamma_{M0} = 1$

$$Shear\ capacity = \frac{2 \cdot \left(\frac{f_u \cdot A_{net}}{\gamma_{M2,1\text{-}1}} + \frac{f_y \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}} \right)}{1000.0}$$

$$= \frac{2 \cdot \left(\frac{410.0 \cdot 190.0}{1.1} + \frac{275.0 \cdot 1730.0}{\sqrt{3} \cdot 1} \right)}{1000.0}$$

$= 691.0\ kN$

Applied member shear, $V_{Ed} = 340.0\ kN$

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

$= \frac{340.0}{691.0}$

$= 0.492041$

$691.0\ kN \geq 340.0\ kN$ **(OK)**

$0.492 \leq 1$ **(OK)**

Connection-gross shear yielding (15). Reference Pg 23

Connection yield stress, $f_y = 275.0\ MPa$

Connection thickness, $t = 10.0\ mm$

Connection depth, $h = 290.0\ mm$

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

$= 2 \cdot 10.0 \cdot 290.0$

$= 5800.0\ mm^2$

$\gamma_{M0} = 1$

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

Connection-gross shear yielding (15). Reference Pg 23 (continued)

$$= \frac{\left(\frac{\left(\frac{275.0 \cdot 5800.0}{\sqrt{3} \cdot 1}\right)}{1.27}\right)}{1000.0}$$

$$= 725.1\ kN$$

Applied member shear, $V_{Ed} = 340.0\ kN$

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

$$= \frac{340.0}{725.1}$$

$$= 0.468901$$

$725.1\ kN \geq 340.0\ kN$ **(OK)**

$0.469 \leq 1$ **(OK)**

Connection-net shear rupture (21). Reference Pg 23

Connection tensile strength, $f_u = 410.0\ MPa$

Bolt rows, $n_1 = 4$

Connection thickness, $t = 10.0\ mm$

Connection depth, $h = 290.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

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

$$= 2 \cdot 10.0 \cdot \left(290.0 - 4 \cdot 22.0\right)$$

$$= 4040.0\ mm^2$$

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

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

$$= \frac{\left(\frac{410.0 \cdot 4040.0}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$$

$$= 869.4\ kN$$

$Shear\ capacity = V_{Rd}$

$$= 869.4\ kN$$

Applied member shear, $V_{Ed} = 340.0\ kN$

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

Connection-net shear rupture (21). Reference Pg 23 (continued)

$$= \frac{340.0}{869.4}$$
$$= 0.391075$$

$869.4\ kN \geq 340.0\ kN$ **(OK)**

$0.391 \leq 1$ **(OK)**

Supported beam-welds (24). Reference Pg 16

Web thickness, $t_w = 9.5\ mm$

Minimum specified weld throat thickness, $a_{min} = 0.4 \cdot t_w$
$$= 0.4 \cdot 9.5$$
$$= 3.8\ mm$$

Minimum specified weld leg size, $s_{min} = \frac{a_{min}}{0.707}$
$$= \frac{3.8}{0.707}$$
$$= 5.4\ mm$$

$(s = 6.0\ mm) \geq (s_{min} = 5.4\ mm)$

Weld is sized to develop the full strength of the beam web.

$Unity = 0$

Connection-bolt shear (1). Reference Pg 21

Number of shear planes, $N_s = 1$

Bolt columns, $n_2 = 2$

Bolt rows, $n_1 = 4$

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

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

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

Not necessary to reduce shear capacity for packing.

Bolt Design capacity, $N_b = \frac{\left(\frac{480.0 \cdot A}{\gamma_{M2,1\text{-}8}}\right) \cdot N_s}{1000.0}$

$$= \frac{\left(\frac{480.0 \cdot 244.8}{1.25}\right) \cdot 1}{1000.0}$$
$$= 94.0\ kN$$

Horizontal shear load, $V_{h,Ed} = 0.0\ kN$

$Shear\ capacity = 0.8 \cdot N_b \cdot n_1 \cdot n_2$
$$= 0.8 \cdot 94.0 \cdot 4 \cdot 2$$
$$= 601.6\ kN$$

Applied member shear, $V_{Ed} = 340.0\ kN$

Connection-bolt shear (1). Reference Pg 21 (continued)

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$
$$= \frac{340.0}{601.6}$$
$$= 0.56516$$

$601.6\ kN \geq 340.0\ kN$ **(OK)**

$0.565 \leq 1$ **(OK)**

Connection-bearing on plate (110). Reference Pg 22

Tensile strength, $f_u = 410.0\ MPa$

Plate thickness, $t_p = 10.0\ mm$

Column edge distance, $e_2 = 30.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Row edge distance, $e_1 = 40.0\ mm$

Bolt diameter, $d = 20.0\ mm$

Bolt columns, $n_2 = 2$

Bolt rows, $n_1 = 4$

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

Length of joint, $l_j = p_{1,total}$
$= 210.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt row spacing, $p_1 = 70.0\ mm$

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

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

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right)$$
$$= min\left(\frac{2.8 \cdot 30.0}{22.0} - 1.7, 2.5\right)$$
$$= 2.11818$$

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

Inner bolt resistance (continued)

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

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

$= 112.6\ kN$

End bolt resistance

$\alpha_d = \dfrac{e_1}{3 \cdot d_0}$

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

$= 0.606061$

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

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

$= 0.606061$

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

$= min\left(\dfrac{2.8 \cdot 30.0}{22.0} - 1.7, 2.5\right)$

$= 2.11818$

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

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

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

$= 84.2\ kN$

Bolt shear resistance

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

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

$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 1 = 75.2\ kN) < (F_{b,Rd,e} = 84.2\ kN)$

$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 1 = 75.2\ kN) < (F_{b,Rd,i} = 112.6\ kN)$

Fastener resistance limited by bolt shear resistance

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

$= min\left(84.2, 112.6, 0.8 \cdot 94.0 \cdot 1\right)$

Connection-bearing on plate (110). Reference Pg 22 (continued)

$= 75.2\ kN$

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

$= 75.2 \cdot 4 \cdot 2$

$= 601.6\ kN$

Applied member shear, $V_{Ed} = 340.0\ kN$

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

$= \dfrac{340.0}{601.6}$

$= 0.56516$

$601.6\ kN \geq 340.0\ kN$ **(OK)**

$0.565 \leq 1$ **(OK)**

Connection-bearing on support (110). Reference Pg 22

Tensile strength, $f_u = 410.0\ MPa$

Plate thickness, $t_p = 6.5\ mm$

Column bolt spacing, $p_2 = 140.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt diameter, $d = 20.0\ mm$

Bolt columns, $n_2 = 2$

Bolt rows, $n_1 = 4$

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

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

$= 210.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt row spacing, $p_1 = 70.0\ mm$

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

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

$= 0.810606$

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

$= min\left(0.810606, \dfrac{800.0}{410.0}, 1\right)$

$= 0.810606$

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

$= min\left(\dfrac{1.4 \cdot 140.0}{22.0} - 1.7, 2.5\right)$

Inner bolt resistance (continued)

$= 2.5$

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

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

$= \dfrac{\left(\dfrac{2.5 \cdot 0.810606 \cdot 410.0 \cdot 20.0 \cdot 6.5}{1.25}\right)}{1000.0}$

$= 87.1\ kN$

End bolt resistance

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

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

$= 1$

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

$= min\left(\dfrac{1.4 \cdot 140.0}{22.0} - 1.7, 2.5\right)$

$= 2.5$

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

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

$= \dfrac{\left(\dfrac{2.5 \cdot 1 \cdot 410.0 \cdot 20.0 \cdot 6.5}{1.25}\right)}{1000.0}$

$= 107.4\ kN$

Bolt shear resistance

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

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

$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 1 = 75.2\ kN) < (F_{b,Rd,e} = 107.4\ kN)$

$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 1 = 75.2\ kN) < (F_{b,Rd,i} = 87.1\ kN)$

Fastener resistance limited by bolt shear resistance

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

$= min\,(107.4, 87.1, 0.8 \cdot 94.0 \cdot 1)$

$= 75.2\ kN$

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

Connection-bearing on support (110). Reference Pg 22 (continued)

$= 75.2 \cdot 4 \cdot 2$

$= 601.6\ kN$

Applied member shear, $V_{Ed} = 340.0\ kN$

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

$= \dfrac{340.0}{601.6}$

$= 0.56516$

$601.6\ kN \geq 340.0\ kN$ **(OK)**

$0.565 \leq 1$ **(OK)**

Supported beam-resistance at notch (364). Reference Pg 18

Yield stress of the supported beam, $f_{y,b1} = 275.0\ MPa$

Thickness of the end plate, $t_p = 10.0\ mm$

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, $d_{nt} = 50.0\ mm$

k distance of the supported beam, $k_{b1} = 26.2\ mm$

Flange thickness of the supported beam, $t_{f,b1} = 16.0\ mm$

Flange width of the supported beam, $b_{f,b1} = 74.0\ mm$

Web thickness of the supported beam, $t_{w,b1} = 9.5\ mm$

Height of the supported beam, $h_{b1} = 412.8\ mm$

Applied member shear, $V_{Ed} = 340.0\ kN$

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

$= max\ (50.0, 0.0)$

$= 50.0\ mm$

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

$= 110.0\ mm$

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

$= max\ (0.0, 0.0)$

$= 0.0\ mm$

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

$= 0.0\ mm$

Top flange is coped.

Distance from supporting face to end of cope, $e = l_{nt,e} + t_p$

$= 110.0 + 10.0$

$= 120.0\ mm$

Area of the Tee section, $A_{Tee} = (h_{b1} - d_{nt,e} - t_{f,b1}) \cdot t_{w,b1} + b_{f,b1} \cdot t_{f,b1}$

$= (412.8 - 50.0 - 16.0) \cdot 9.5 + 74.0 \cdot 16.0$

Supported beam-resistance at notch (364). Reference Pg 18 (continued)

$= 4478.6\ mm^2$

Shear area at the notch for single notched beam, $A_{v,N} = A_{Tee} - b_{f,b1} \cdot t_{f,b1} + \frac{k_{b1} \cdot t_{f,b1}}{2}$

$= 4478.6 - 74.0 \cdot 16.0 + \frac{26.2 \cdot 16.0}{2}$

$= 3504.2\ mm^2$

Elastic modulus of the Tee section at the notch, $W_{el,N,y} = 278763.8\ mm^3$

$\gamma_{M0} = 1$

Shear resistance at the notch for single notched beam, $V_{pl,N,Rd} = \frac{\left(\frac{A_{v,N} \cdot f_{y,b1}}{1.73205 \cdot \gamma_{M0}}\right)}{1000.0}$

$= \frac{\left(\frac{3504.2 \cdot 275.0}{1.73205 \cdot 1}\right)}{1000.0}$

$= 556.4\ kN$

$\gamma_{M0} = 1$

Moment resistance of a single notched beam at the notch in the presence of shear, $M_{v,N,Rd} = \frac{\left(\frac{f_{y,b1} \cdot W_{el,N,y}}{\gamma_{M0}}\right) \cdot \left(1 - \left(\frac{2 \cdot V_{Ed}}{V_{pl,N,Rd}} - \right.\right.}{1000000.0}$

$= \frac{\left(\frac{275.0 \cdot 278763.8}{1}\right) \cdot \left(1 - \left(\frac{2 \cdot 340.0}{556.4} - 1\right)^2\right)}{1000000.0}$

$= 72.9\ kN \cdot m$

Shear resistance at the end of the beam, $V_{Rd} = \left(\frac{M_{v,N,Rd}}{e}\right) \cdot 1000.0$

$= \left(\frac{72.9}{120.0}\right) \cdot 1000.0$

$= 607.3\ kN$

$Shear\ capacity = V_{Rd}$

$= 607.3\ kN$

Applied member shear, $V_{Ed} = 340.0\ kN$

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

$= \frac{340.0}{607.3}$

$= 0.559856$

$607.3\ kN \geq 340.0\ kN$ **(OK)**

$0.560 \leq 1$ **(OK)**

Connection-in-plane bending (365). Reference Pg 23

Not necessary to check in-plane bending.

$Unity = 0$

Supporting member-shear (409). Reference Pg 24

Tensile strength of the supporting member, $f_{u,2} = 410.0\ MPa$

Yield stress of the supporting member, $f_{y,2} = 265.0\ MPa$

Supporting thickness, $t_2 = 6.5\ mm$

Bolt diameter, $d = 20.0\ mm$

Edge distance to bottom of support, $e_{1,b} = 317.2\ mm$

Edge distance to top of support, $e_{1,t} = 90.0\ mm$

Bolt gage, $p_3 = 140.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Bolt rows, $n_1 = 4$

$e_t = min\ (e_{1,t}, 5 \cdot d)$

$= min\ (90.0, 5 \cdot 20.0)$

$= 90.0\ mm$

$e_b = min\ \left(e_{1,b}, \frac{p_3}{2}, 5 \cdot d\right)$

$= min\ \left(317.2, \frac{140.0}{2}, 5 \cdot 20.0\right)$

$= 70.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

Gross shear area, $A_{gv} = t_2 \cdot (e_t + (n_1 - 1) \cdot p_1 + e_b)$

$= 6.5 \cdot (90.0 + (4 - 1) \cdot 70.0 + 70.0)$

$= 2423.5\ mm^2$

Net shear area, $A_{nv} = A_{gv} - n_1 \cdot d_0 \cdot t_2$

$= 2423.5 - 4 \cdot 22.0 \cdot 6.5$

$= 1847.1\ mm^2$

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

$\gamma_{M0} = 1$

Minimum shear resistance of the supporting member, $V_{Rd,min} = \dfrac{2 \cdot min\left(\dfrac{f_{y,2} \cdot A_{gv}}{\sqrt{3} \cdot \gamma_{M0}}, \dfrac{f_{u,2} \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M2,1\text{-}1}}\right)}{1000.0}$

$$= \frac{2 \cdot min\left(\frac{265.0 \cdot 2423.5}{\sqrt{3} \cdot 1}, \frac{410.0 \cdot 1847.1}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$$

$= 741.6\ kN$

$Shear\ capacity = V_{Rd,min}$

Supporting member-shear (409). Reference Pg 24 (continued)

$= 741.6\ kN$

Applied member shear, $V_{Ed} = 340.0\ kN$

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

$$= \frac{340.0}{741.6}$$

$$= 0.458469$$

$741.6\ kN \geq 340.0\ kN$ **(OK)**

$0.458 \leq 1$ **(OK)**

Beam B_12 [11]

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:	4981
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:	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:	End plate
	Wide gage, Partial depth end plate
	Welded extended tee: No
	Safety Automatic
	Expand Vertical bolt spacing: Auto
	Use web extension: If Required
Elevation:	0
Minus Dim:	6.5 mm (AUTO)
Mtrl Setback:	18.5 mm (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	550.0 kN
Moment:	0.0 kN·m (AUTO)
Tension:	0.0 kN
Compress:	0.0 kN
Tying:	0.0 kN (AUTO)

B_12 [11] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Beam:	B_11 [12]
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:	S275a
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: 550.0 kN

Design load notes

- Reaction has been input
- Design reaction is 53.0 % of the allowable uniform steel beam load.

Connection summary

- END PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	S275
	Tensile strength, f_u:	410.0 MPa
	Yield stress, f_y:	275.0 MPa
	Thickness, t:	12.0 mm
	Depth, h:	430.0 mm
	Width, b:	200.0 mm
Welds:	**Weld type:**	Double fillet
	Weld leg size, s:	6.0 mm
	Weld metal strength, F_{exx}:	0.0 MPa
	Total effective weld throat, a_e:	8.5 mm
Bolts:	**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 gage, p_3:	140.0 mm
	Row edge distance, e_1:	40.0 mm
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit:	3
(at dd) Not Locked:	284

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

Expanded design calculation

Supported beam-web shear (8). Reference Pg 17

Beam yield stress, $f_{y,b1} = 275.0\ MPa$

Connection depth, $h_p = 430.0\ mm$

Web thickness, $t_w = 10.1\ mm$

Shear Area, $A_g = 0.9 \cdot h_p \cdot t_w$

$= 0.9 \cdot 430.0 \cdot 10.1$

$= 3908.7\ mm^2$

$\gamma_{M0} = 1$

Allowable shear capacity, $R_v = \dfrac{\left(\dfrac{f_{y,b1} \cdot A_g}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{275.0 \cdot 3908.7}{\sqrt{3} \cdot 1}\right)}{1000.0}$

$= 620.6\ kN$

Applied member shear, $V_{Ed} = 550.0\ kN$

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

$= \dfrac{550.0}{620.6}$

$= 0.886255$

$Shear\ capacity = R_v$

$= 620.6\ kN$

$620.6\ kN \geq 550.0\ kN$ **(OK)**

$0.886 \leq 1$ **(OK)**

Supported beam-stability at notch (41). Reference Pg 19

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, $d_{nb,e} = max\ (d_{nb}, 0.0)$

Supported beam-stability at notch (41). Reference Pg 19 (continued)

$= max\ (0.0{,}0.0)$

$= 0.0\ mm$

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.

Connection-block tearing (252). Reference Pg 23

Yield stress, $f_y = 275.0\ MPa$

Tensile strength, $f_u = 410.0\ MPa$

Connection thickness, $t = 12.0\ mm$

Bolt column spacing, $p_2 = 0.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt columns, $n_2 = 1$

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

Horizontal edge distance, $e_2 = 30.0\ mm$

Vertical edge distance, $e_1 = 40.0\ mm$

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, $A_{gv} = t \cdot (p_{1,total} + e_1)$

$= 12.0 \cdot (350.0 + 40.0)$

$= 4680.0\ mm^2$

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

$= 12.0 \cdot (350.0 + 40.0) - 12.0 \cdot (6 - 0.5) \cdot 22.0$

$= 3228.0\ mm^2$

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

$= 12.0 \cdot (0.0 \cdot (1 - 1) + 30.0)$

$= 360.0\ mm^2$

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

Calculate block shear capacity of each side (continued)

$= 12.0 \cdot (0.0 \cdot (1 - 1) + 30.0) - 12.0 \cdot (1 - 0.5) \cdot 22.0$

$= 228.0\ mm^2$

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

$\gamma_{M0} = 1$

$$Shear\ capacity = \frac{2 \cdot \left(\frac{f_u \cdot A_{net}}{\gamma_{M2,1\text{-}1}} + \frac{f_y \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$$

$$= \frac{2 \cdot \left(\frac{410.0 \cdot 228.0}{1.1} + \frac{275.0 \cdot 3228.0}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$= 1195.0\ kN$

Applied member shear, $V_{Ed} = 550.0\ kN$

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

$= \frac{550.0}{1195.0}$

$= 0.460251$

$1195.0\ kN \geq 550.0\ kN$ **(OK)**

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

Connection-gross shear yielding (15). Reference Pg 23

Connection yield stress, $f_y = 275.0\ MPa$

Connection thickness, $t = 12.0\ mm$

Connection depth, $h = 430.0\ mm$

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

$= 2 \cdot 12.0 \cdot 430.0$

$= 10320.0\ mm^2$

$\gamma_{M0} = 1$

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

Connection-gross shear yielding (15). Reference Pg 23 (continued)

$$= \frac{\left(\frac{\left(\frac{275.0 \cdot 10320.0}{\sqrt{3} \cdot 1}\right)}{1.27}\right)}{1000.0}$$

$$= 1290.2\ kN$$

Applied member shear, $V_{Ed} = 550.0\ kN$

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

$$= \frac{550.0}{1290.2}$$

$$= 0.426291$$

$1290.2\ kN \geq 550.0\ kN$ **(OK)**

$0.426 \leq 1$ **(OK)**

Connection-net shear rupture (21). Reference Pg 23

Connection tensile strength, $f_u = 410.0\ MPa$

Bolt rows, $n_1 = 6$

Connection thickness, $t = 12.0\ mm$

Connection depth, $h = 430.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

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

$$= 2 \cdot 12.0 \cdot (430.0 - 6 \cdot 22.0)$$

$$= 7152.0\ mm^2$$

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

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

$$= \frac{\left(\frac{410.0 \cdot 7152.0}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$$

$$= 1539.1\ kN$$

$Shear\ capacity = V_{Rd}$

$$= 1539.1\ kN$$

Applied member shear, $V_{Ed} = 550.0\ kN$

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

Connection-net shear rupture (21). Reference Pg 23 (continued)

$= \frac{550.0}{1539.1}$

$= 0.357352$

$1539.1\ kN \geq 550.0\ kN$ **(OK)**

$0.357 \leq 1$ **(OK)**

Supported beam-welds (24). Reference Pg 16

Web thickness, $t_w = 10.1\ mm$

Minimum specified weld throat thickness, $a_{min} = 0.4 \cdot t_w$

$= 0.4 \cdot 10.1$

$= 4.0\ mm$

Minimum specified weld leg size, $s_{min} = \frac{a_{min}}{0.707}$

$= \frac{4.0}{0.707}$

$= 5.7\ mm$

$(s = 6.0\ mm) \geq (s_{min} = 5.7\ mm)$

Weld is sized to develop the full strength of the beam web.

$Unity = 0$

Connection-bolt shear (1). Reference Pg 21

Number of shear planes, $N_s = 1$

Bolt columns, $n_2 = 2$

Bolt rows, $n_1 = 6$

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

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

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

Not necessary to reduce shear capacity for packing.

Bolt Design capacity, $N_b = \frac{\left(\frac{480.0 \cdot A}{\gamma_{M2,1\text{-}8}}\right) \cdot N_s}{1000.0}$

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

$= 94.0\ kN$

Horizontal shear load, $V_{h,Ed} = 0.0\ kN$

$Shear\ capacity = 0.8 \cdot N_b \cdot n_1 \cdot n_2$

$= 0.8 \cdot 94.0 \cdot 6 \cdot 2$

$= 902.4\ kN$

Applied member shear, $V_{Ed} = 550.0\ kN$

Connection-bolt shear (1). Reference Pg 21 (continued)

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$
$$= \frac{550.0}{902.4}$$
$$= 0.609486$$

$902.4\ kN \geq 550.0\ kN$ **(OK)**

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

Connection-bearing on plate (110). Reference Pg 22

Tensile strength, $f_u = 410.0\ MPa$

Plate thickness, $t_p = 12.0\ mm$

Column edge distance, $e_2 = 30.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Row edge distance, $e_1 = 40.0\ mm$

Bolt diameter, $d = 20.0\ mm$

Bolt columns, $n_2 = 2$

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, $p_1 = 70.0\ mm$

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

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

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right)$$
$$= min\left(\frac{2.8 \cdot 30.0}{22.0} - 1.7, 2.5\right)$$
$$= 2.11818$$

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

Inner bolt resistance (continued)

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

$= \dfrac{\left(\dfrac{2.11818 \cdot 0.810606 \cdot 410.0 \cdot 20.0 \cdot 12.0}{1.25}\right)}{1000.0}$

$= 135.2\ kN$

End bolt resistance

$\alpha_d = \dfrac{e_1}{3 \cdot d_0}$

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

$= 0.606061$

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

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

$= 0.606061$

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

$= min\left(\dfrac{2.8 \cdot 30.0}{22.0} - 1.7, 2.5\right)$

$= 2.11818$

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

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

$= \dfrac{\left(\dfrac{2.11818 \cdot 0.606061 \cdot 410.0 \cdot 20.0 \cdot 12.0}{1.25}\right)}{1000.0}$

$= 101.1\ kN$

Bolt shear resistance

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

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

$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 0.9875 = 74.3\ kN) < (F_{b,Rd,e} = 101.1\ kN)$

$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 0.9875 = 74.3\ kN) < (F_{b,Rd,i} = 135.2\ kN)$

Fastener resistance limited by bolt shear resistance

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

$= min\left(101.1, 135.2, 0.8 \cdot 94.0 \cdot 0.9875\right)$

Connection-bearing on plate (110). Reference Pg 22 (continued)

$= 74.3\ kN$

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

$= 74.3 \cdot 6 \cdot 2$

$= 891.1\ kN$

Applied member shear, $V_{Ed} = 550.0\ kN$

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

$= \frac{550.0}{891.1}$

$= 0.617215$

$891.1\ kN \geq 550.0\ kN$ **(OK)**

$0.617 \leq 1$ **(OK)**

Connection-bearing on support (110). Reference Pg 22

Tensile strength, $f_u = 410.0\ MPa$

Plate thickness, $t_p = 6.5\ mm$

Column bolt spacing, $p_2 = 140.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Bolt diameter, $d = 20.0\ mm$

Bolt columns, $n_2 = 2$

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, $p_1 = 70.0\ mm$

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

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

$= 0.810606$

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

$= min\left(0.810606, \frac{800.0}{410.0}, 1\right)$

$= 0.810606$

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

$= min\left(\frac{1.4 \cdot 140.0}{22.0} - 1.7, 2.5\right)$

Inner bolt resistance (continued)

$= 2.5$

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

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

$= \dfrac{\left(\dfrac{2.5 \cdot 0.810606 \cdot 410.0 \cdot 20.0 \cdot 6.5}{1.25}\right)}{1000.0}$

$= 87.1\ kN$

End bolt resistance

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

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

$= 1$

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

$= min\left(\dfrac{1.4 \cdot 140.0}{22.0} - 1.7, 2.5\right)$

$= 2.5$

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

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

$= \dfrac{\left(\dfrac{2.5 \cdot 1 \cdot 410.0 \cdot 20.0 \cdot 6.5}{1.25}\right)}{1000.0}$

$= 107.4\ kN$

Bolt shear resistance

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

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

$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 0.9875 = 74.3\ kN) < (F_{b,Rd,e} = 107.4\ kN)$

$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 0.9875 = 74.3\ kN) < (F_{b,Rd,i} = 87.1\ kN)$

Fastener resistance limited by bolt shear resistance

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

$= min\ (107.4, 87.1, 0.8 \cdot 94.0 \cdot 0.9875)$

$= 74.3\ kN$

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

Connection-bearing on support (110). Reference Pg 22 (continued)

$= 74.3 \cdot 6 \cdot 2$

$= 891.1\ kN$

Applied member shear, $V_{Ed} = 550.0\ kN$

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

$= \dfrac{550.0}{891.1}$

$= 0.617215$

$891.1\ kN \geq 550.0\ kN$ **(OK)**

$0.617 \leq 1$ **(OK)**

Supported beam-resistance at notch (364). Reference Pg 18

Yield stress of the supported beam, $f_{y,b1} = 275.0\ MPa$

Thickness of the end plate, $t_p = 12.0\ mm$

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, $d_{nt} = 50.0\ mm$

k distance of the supported beam, $k_{b1} = 28.3\ mm$

Flange thickness of the supported beam, $t_{f,b1} = 15.6\ mm$

Flange width of the supported beam, $b_{f,b1} = 209.3\ mm$

Web thickness of the supported beam, $t_{w,b1} = 10.1\ mm$

Height of the supported beam, $h_{b1} = 533.1\ mm$

Applied member shear, $V_{Ed} = 550.0\ kN$

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

$= max\ (50.0, 0.0)$

$= 50.0\ mm$

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

$= 110.0\ mm$

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

$= max\ (0.0, 0.0)$

$= 0.0\ mm$

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

$= 0.0\ mm$

Top flange is coped.

Distance from supporting face to end of cope, $e = l_{nt,e} + t_p$

$= 110.0 + 12.0$

$= 122.0\ mm$

Area of the Tee section, $A_{Tee} = (h_{b1} - d_{nt,e} - t_{f,b1}) \cdot t_{w,b1} + b_{f,b1} \cdot t_{f,b1}$

$= (533.1 - 50.0 - 15.6) \cdot 10.1 + 209.3 \cdot 15.6$

Supported beam-resistance at notch (364). Reference Pg 18 (continued)

$= 7986.8\ mm^2$

Shear area at the notch for single notched beam, $A_{v,N} = A_{Tee} - b_{f,b1} \cdot t_{f,b1} + \frac{k_{b1} \cdot t_{f,b1}}{2}$

$= 7986.8 - 209.3 \cdot 15.6 + \frac{28.3 \cdot 15.6}{2}$

$= 4942.5\ mm^2$

Elastic modulus of the Tee section at the notch, $W_{el,N,y} = 597565.2\ mm^3$

$\gamma_{M0} = 1$

Shear resistance at the notch for single notched beam, $V_{pl,N,Rd} = \frac{\left(\frac{A_{v,N} \cdot f_{y,b1}}{1.73205 \cdot \gamma_{M0}}\right)}{1000.0}$

$= \frac{\left(\frac{4942.5 \cdot 275.0}{1.73205 \cdot 1}\right)}{1000.0}$

$= 784.7\ kN$

$\gamma_{M0} = 1$

Moment resistance of a single notched beam at the notch in the presence of shear, $M_{v,N,Rd} = \frac{\left(\frac{f_{y,b1} \cdot W_{el,N,y}}{\gamma_{M0}}\right) \cdot \left(1 - \left(\frac{2 \cdot V_{Ed}}{V_{pl,N,Rd}} - \right.\right.}{1000000.0}$

$= \frac{\left(\frac{275.0 \cdot 597565.2}{1}\right) \cdot \left(1 - \left(\frac{2 \cdot 550.0}{784.7} - 1\right)^2\right)}{1000000.0}$

$= 137.8\ kN \cdot m$

Shear resistance at the end of the beam, $V_{Rd} = \left(\frac{M_{v,N,Rd}}{e}\right) \cdot 1000.0$

$= \left(\frac{137.8}{122.0}\right) \cdot 1000.0$

$= 1129.5\ kN$

$Shear\ capacity = V_{Rd}$

$= 1129.5\ kN$

Applied member shear, $V_{Ed} = 550.0\ kN$

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

$= \frac{550.0}{1129.5}$

$= 0.486942$

$1129.5\ kN \geq 550.0\ kN$ **(OK)**

$0.487 \leq 1$ **(OK)**

Connection-in-plane bending (365). Reference Pg 23

Not necessary to check in-plane bending.

$Unity = 0$

Supporting member-shear (409). Reference Pg 24

Tensile strength of the supporting member, $f_{u,2} = 410.0\ MPa$

Yield stress of the supporting member, $f_{y,2} = 265.0\ MPa$

Supporting thickness, $t_2 = 6.5\ mm$

Bolt diameter, $d = 20.0\ mm$

Edge distance to bottom of support, $e_{1,b} = 177.2\ mm$

Edge distance to top of support, $e_{1,t} = 90.0\ mm$

Bolt gage, $p_3 = 140.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Bolt rows, $n_1 = 6$

$e_t = min\left(e_{1,t}, 5 \cdot d\right)$

$= min\left(90.0, 5 \cdot 20.0\right)$

$= 90.0\ mm$

$e_b = min\left(e_{1,b}, \frac{p_3}{2}, 5 \cdot d\right)$

$= min\left(177.2, \frac{140.0}{2}, 5 \cdot 20.0\right)$

$= 70.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

Gross shear area, $A_{gv} = t_2 \cdot \left(e_t + \left(n_1 - 1\right) \cdot p_1 + e_b\right)$

$= 6.5 \cdot \left(90.0 + \left(6 - 1\right) \cdot 70.0 + 70.0\right)$

$= 3340.5\ mm^2$

Net shear area, $A_{nv} = A_{gv} - n_1 \cdot d_0 \cdot t_2$

$= 3340.5 - 6 \cdot 22.0 \cdot 6.5$

$= 2475.9\ mm^2$

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

$\gamma_{M0} = 1$

Minimum shear resistance of the supporting member, $V_{Rd,min} = \dfrac{2 \cdot min\left(\dfrac{f_{y,2} \cdot A_{gv}}{\sqrt{3} \cdot \gamma_{M0}}, \dfrac{f_{u,2} \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M2,1\text{-}1}}\right)}{1000.0}$

$$= \frac{2 \cdot min\left(\dfrac{265.0 \cdot 3340.5}{\sqrt{3} \cdot 1}, \dfrac{410.0 \cdot 2475.9}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$$

$= 1022.2\ kN$

$Shear\ capacity = V_{Rd,min}$

Supporting member-shear (409). Reference Pg 24 (continued)

$= 1022.2\ kN$

Applied member shear, $V_{Ed} = 550.0\ kN$

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

$$= \frac{550.0}{1022.2}$$

$$= 0.538056$$

$1022.2\ kN \geq 550.0\ kN$ **(OK)**

$0.538 \leq 1$ **(OK)**

Results summary

End Plate on right end of Beam B_10 [10]

Design calculation summary for member [10], right end

Desc. Ref.:	Calc. Num.	Unity Ratio	Resistance	Design Force	Green book ref.
4:	(8)	0.864	393.7 kN	340.0 kN	Pg 17
5:	(364)	0.560	607.3 kN	340.0 kN	Pg 18
8(i):	(1)	0.565	601.6 kN	340.0 kN	Pg 21
8(ii):	(110)	0.565	601.6 kN	340.0 kN	Pg 22
8(iii):	(110)	0.565	601.6 kN	340.0 kN	Pg 22
9(i):	(15)	0.469	725.1 kN	340.0 kN	Pg 23
9(ii):	(21)	0.391	869.4 kN	340.0 kN	Pg 23
9(iii):	(252)	0.492	691.0 kN	340.0 kN	Pg 23
10:	(409)	0.458	741.6 kN	340.0 kN	Pg 24

Check number reference

4:	Supported beam-web shear
5:	Supported beam-resistance at notch
8(i):	Connection-bolt shear
8(ii):	Connection-bearing on plate
8(iii):	Connection-bearing on support
9(i):	Connection-gross shear yielding
9(ii):	Connection-net shear rupture
9(iii):	Connection-block tearing
10:	Supporting member-shear

Connection strength

	Unity ratio:
Shear:	0.864

Notes and conclusions

- Effective weld length = plate length - 2 * (weld size) or beam d distance, whichever is less.
- CONNECTION IS OK
    - Design resistance equals or exceeds design forces.
- Weld size satisfies basic requirements of Check 2 from the Green Book.

End Plate on left end of Beam B_12 [11]

Design calculation summary for member [11], left end

Desc. Ref.:	Calc. Num.	Unity Ratio	Resistance	Design Force	Green book ref.
4:	(8)	0.886	620.6 kN	550.0 kN	Pg 17
5:	(364)	0.487	1129.5 kN	550.0 kN	Pg 18
8(i):	(1)	0.609	902.4 kN	550.0 kN	Pg 21
8(ii):	(110)	0.617	891.1 kN	550.0 kN	Pg 22
8(iii):	(110)	0.617	891.1 kN	550.0 kN	Pg 22
9(i):	(15)	0.426	1290.2 kN	550.0 kN	Pg 23
9(ii):	(21)	0.357	1539.1 kN	550.0 kN	Pg 23
9(iii):	(252)	0.460	1195.0 kN	550.0 kN	Pg 23
10:	(409)	0.538	1022.2 kN	550.0 kN	Pg 24

Check number reference

4:	Supported beam-web shear
5:	Supported beam-resistance at notch
8(i):	Connection-bolt shear
8(ii):	Connection-bearing on plate
8(iii):	Connection-bearing on support
9(i):	Connection-gross shear yielding
9(ii):	Connection-net shear rupture
9(iii):	Connection-block tearing
10:	Supporting member-shear

Connection strength

	Unity ratio:
Shear:	0.886

Notes and conclusions

- Effective weld length = plate length - 2 * (weld size) or beam d distance, whichever is less.
- CONNECTION IS OK
    - Design resistance equals or exceeds design forces.
- Weld size satisfies basic requirements of Check 2 from the Green Book.

Additional notes

Example 1 - Partial depth end plate, page 32,
P358 Simple Joints.pdf