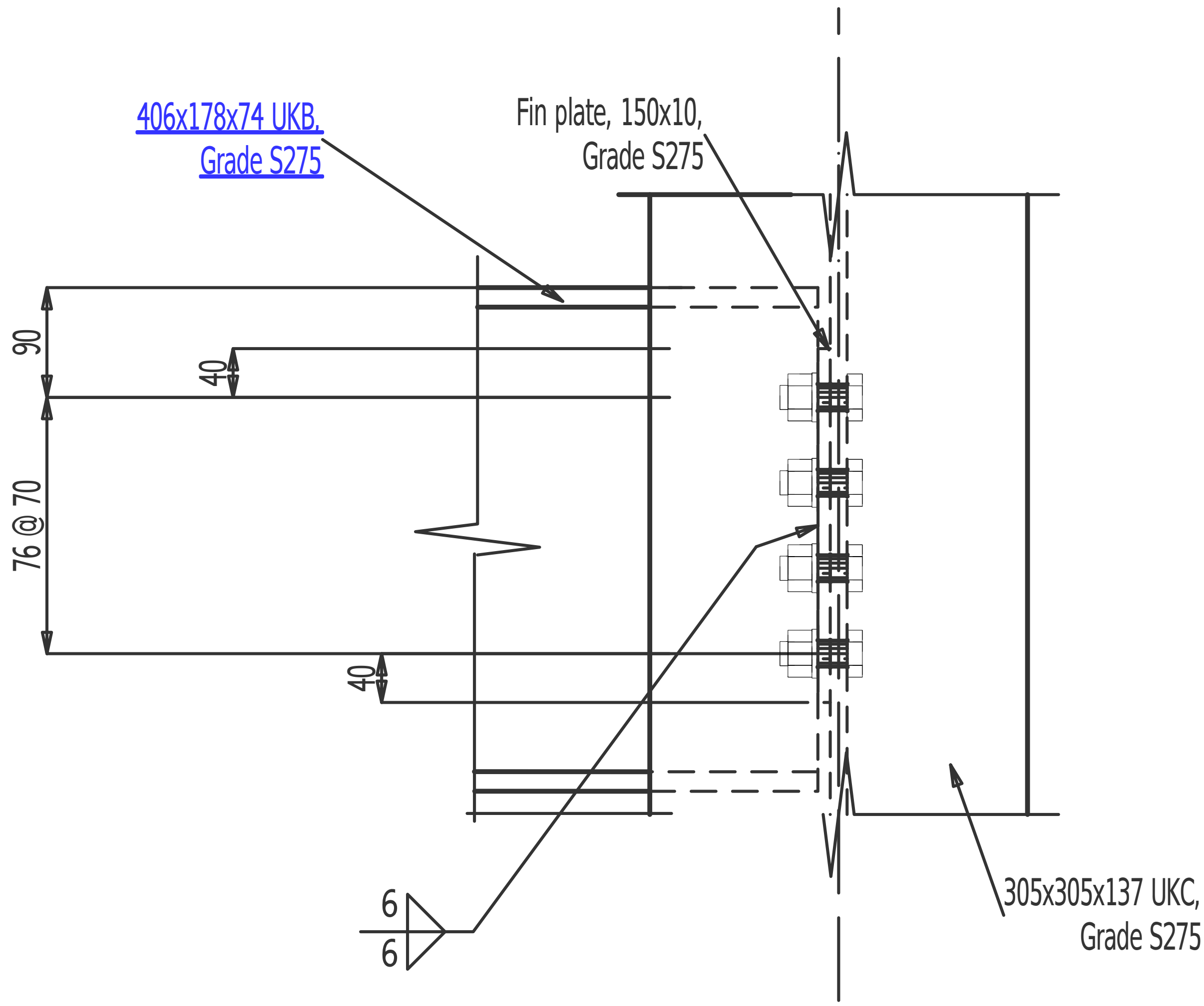
SDS2 Steel Connection Design: Connection Cube Report

Cube: Partial depth end plate-tying
Revision: 0
Project: 0854_Validation_Eurocode_3UK
Engineer:
Fabricator: Validation_Eurocode_3UK

Generated by SDS2 x 2025.07 on Thursday, Mar 13, 2025

Partial depth end plate-tying [3] at X=17500, Y=-5000 Elev=2294



ELEVATION VIEW

Beam B_14 [15]

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:	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:	2500
Minus Dim:	6.9 mm (AUTO)
Mtrl Setback:	16.9 mm (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	10.0 kN
Moment:	0.0 kN·m (AUTO)
Tension:	0.0 kN
Compress:	0.0 kN
Tying:	175.0 kN (AUTO)

B_14 [15] Connection strength check: RIGHT END

Member end summary

Connecting nodes

Node 1

Column:	C_9 [14]
Section size:	UC305x305x137
End 0 elevation:	0 mm
End 1 elevation:	5000 mm
Framing condition:	Web of Column
Material grade:	S275a
Distance between web toes of fillets, d:	246.7 mm
Supporting member thickness, t_2:	13.8 mm

Factored loads

***Shear*:**	10.0 kN
***Tying force*:**	175.0 kN

Design load notes

- Reaction has been input
- Design reaction is 1.5 % 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

Connection details (continued)

	Yield stress, f_y:	275.0 MPa
	Thickness, t:	10.0 mm
	Depth, h:	290.0 mm
	Width, b:	150.0 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:	90.0 mm
	Row edge distance, e_1:	40.0 mm
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit:	5
(at dd) Not Locked:	282

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, $V_{Ed} = 10.0\ kN$

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

$= \dfrac{10.0}{393.7}$

$= 0.0254018$

$Shear\ capacity = R_v$

$= 393.7\ kN$

$393.7\ kN \geq 10.0\ kN$ **(OK)**

$0.025 \leq 1$ **(OK)**

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

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

$= 190.0\ mm^2$

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

$\gamma_{M0} = 1$

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

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

$= 691.0\ kN$

Applied member shear, $V_{Ed} = 10.0\ kN$

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

$$= \frac{10.0}{691.0}$$

$= 0.0144718$

$691.0\ kN \geq 10.0\ kN$ **(OK)**

$0.014 \leq 1$ **(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, $A_{gv} = 2 \cdot t \cdot h$

$= 2 \cdot 10.0 \cdot 290.0$

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

$= 5800.0\ mm^2$

$\gamma_{M0} = 1$

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

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

$= 725.1\ kN$

Applied member shear, $V_{Ed} = 10.0\ kN$

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

$= \frac{10.0}{725.1}$

$= 0.0137912$

$725.1\ kN \geq 10.0\ kN$ **(OK)**

$0.014 \leq 1$ **(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, $A_{nv} = 2 \cdot t \cdot (h - n_1 \cdot d_0)$

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

$= 4040.0\ mm^2$

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

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

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, $V_{Ed} = 10.0\ kN$

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

$= \frac{10.0}{869.4}$

$= 0.0115022$

$869.4\ kN \geq 10.0\ kN$ **(OK)**

$0.012 \leq 1$ **(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.

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

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

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

$= 94.0\ kN$

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, $V_{Ed} = 10.0\ kN$

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

$= \dfrac{10.0}{601.6}$

$= 0.0166224$

$601.6\ kN \geq 10.0\ kN$ **(OK)**

$0.017 \leq 1$ **(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, $p_1 = 70.0\ mm$

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

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

$= 0.810606$

Inner bolt resistance (continued)

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

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

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

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

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

$$= 112.6\ kN$$

End bolt resistance

$$\alpha_d = \frac{e_1}{3 \cdot d_0}$$
$$= \frac{40.0}{3 \cdot 22.0}$$
$$= 0.606061$$

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

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

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

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

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

End bolt resistance (continued)

$= 84.2\ kN$

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

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

$= 75.2\ kN$

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

$= 75.2 \cdot 4 \cdot 2$

$= 601.6\ kN$

Applied member shear, $V_{Ed} = 10.0\ kN$

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

$= \dfrac{10.0}{601.6}$

$= 0.0166224$

$601.6\ kN \geq 10.0\ kN$ **(OK)**

$0.017 \leq 1$ **(OK)**

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

Tensile strength, $f_u = 410.0\ MPa$

Plate thickness, $t_p = 13.8\ mm$

Column bolt spacing, $p_2 = 90.0\ mm$

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, $p_1 = 70.0\ mm$

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

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

Inner bolt resistance (continued)

$$= 0.810606$$

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

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

$$= 0.810606$$

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

$$= min\left(\frac{1.4 \cdot 90.0}{22.0} - 1.7, 2.5\right)$$

$$= 2.5$$

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

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

$$= \frac{\left(\dfrac{2.5 \cdot 0.810606 \cdot 410.0 \cdot 20.0 \cdot 13.8}{1.25}\right)}{1000.0}$$

$$= 183.5\ kN$$

End bolt resistance

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

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

$$= 1$$

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

$$= min\left(\frac{1.4 \cdot 90.0}{22.0} - 1.7, 2.5\right)$$

$$= 2.5$$

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

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

$$= \frac{\left(\dfrac{2.5 \cdot 1 \cdot 410.0 \cdot 20.0 \cdot 13.8}{1.25}\right)}{1000.0}$$

$$= 226.3\ kN$$

Bolt shear resistance

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

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

$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 1 = 75.2\ kN) < (F_{b,Rd,e} = 226.3\ kN)$

$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 1 = 75.2\ kN) < (F_{b,Rd,i} = 183.5\ kN)$

Fastener resistance limited by bolt shear resistance

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

$= min\ (226.3, 183.5, 0.8 \cdot 94.0 \cdot 1)$

$= 75.2\ kN$

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

$= 75.2 \cdot 4 \cdot 2$

$= 601.6\ kN$

Applied member shear, $V_{Ed} = 10.0\ kN$

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

$$= \frac{10.0}{601.6}$$

$= 0.0166224$

$601.6\ kN \geq 10.0\ kN$ **(OK)**

$0.017 \leq 1$ **(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, $t_2 = 13.8\ mm$

Bolt diameter, $d = 20.0\ mm$

Edge distance to top of support, $e_{1,t} = 2590.0\ mm$

Bolt gage, $p_3 = 90.0\ mm$

Bolt row spacing, $p_1 = 70.0\ mm$

Bolt rows, $n_1 = 4$

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

$= min\ (2590.0, 5 \cdot 20.0)$

$= 100.0\ mm$

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

$$= min\ \left(\frac{90.0}{2}, 5 \cdot 20.0\right)$$

$= 45.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

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

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

$= 13.8 \cdot (100.0 + (4 - 1) \cdot 70.0 + 45.0)$

$= 4899.0\ mm^2$

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

$= 4899.0 - 4 \cdot 22.0 \cdot 13.8$

$= 3684.6\ mm^2$

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

$\gamma_{M0} = 1$

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

$$= \frac{2 \cdot min\left(\dfrac{265.0 \cdot 4899.0}{\sqrt{3} \cdot 1}, \dfrac{410.0 \cdot 3684.6}{\sqrt{3} \cdot 1.1}\right)}{1000.0}$$

$= 1499.1\ kN$

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

$= 1499.1\ kN$

Applied member shear, $V_{Ed} = 10.0\ kN$

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

$= \dfrac{10.0}{1499.1}$

$= 0.00667067$

$1499.1\ kN \geq 10.0\ kN$ **(OK)**

$0.007 \leq 1$ **(OK)**

Tying resistance-plate and bolts (373). Reference Pg 26

Size of the weld on the beam web, $s_w = 6.0\ mm$

End distance of the end plate, $e_1 = 40.0\ mm$

Ultimate tensile strength of end plate, $f_{u,p} = 410.0\ MPa$

Bolt gage, $p_3 = 90.0\ mm$

Bolt row spacing, $p_1 = 70\ mm$

Width of the end plate, $w_p = 150.0\ mm$

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

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

Bolt row, $n_1 = 4$

Column edge distance on end plate, $e_2 = \dfrac{(w_p - p_3)}{2}$

Tying resistance-plate and bolts (373). Reference Pg 26 (continued)

$= \frac{(150.0 - 90.0)}{2}$

$= 30.0\ mm$

$e_{min} = e_2$

$= 30.0\ mm$

$m = \frac{(p_3 - t_{w,b1} - 2 \cdot 0.8 \cdot s_w)}{2}$

$= \frac{(90.0 - 9.5 - 2 \cdot 0.8 \cdot 6.0)}{2}$

$= 35.4\ mm$

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

$= min\ (30.0, 1.25 \cdot 35.4)$

$= 30.0\ mm$

Hole diameter, $d_0 = 22.0\ mm$

$e_{1,A} = min\ \left(e_1, 0.5 \cdot (p_3 - t_{w,b1} - 2 \cdot s_w) + \frac{d_0}{2}\right)$

$= min\ \left(40.0, 0.5 \cdot (90.0 - 9.5 - 2 \cdot 6.0) + \frac{22.0}{2}\right)$

$= 40.0\ mm$

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

$p_{1,A} = min\ (p_{1,total}, (p_3 - t_{w,b1} - 2 \cdot s_w + d_0) \cdot n_1)$

$= min\ (210.0, (90.0 - 9.5 - 2 \cdot 6.0 + 22.0) \cdot 4)$

$= 210.0\ mm$

Effective length, $l_{eff} = 2 \cdot e_{1,A} + p_{1,A}$

$= 2 \cdot 40.0 + 210.0$

$= 290.0\ mm$

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

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

$= \frac{33.0}{4}$

$= 8.2\ mm$

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

$k_2 = 0.9$

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

$\gamma_{Mu} = 1.1$

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

Tying resistance-plate and bolts (373). Reference Pg 26 (continued)

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

$$= 160.2\ kN$$

$$\gamma_{Mu} = 1.1$$

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

$$= \frac{\left(\frac{0.25 \cdot 290.0 \cdot 10.0^2 \cdot 410.0}{1.1}\right)}{1000000.0}$$

$$= 2.7\ kN \cdot m$$

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

$$= 2.7\ kN \cdot m$$

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

$$= \left(\frac{(8 \cdot 30.0 - 2 \cdot 8.2) \cdot 2.7}{(2 \cdot 35.4 \cdot 30.0 - 8.2 \cdot (35.4 + 30.0))}\right) \cdot 1000.0$$

$$= 380.6\ kN$$

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

$$= \left(\frac{\left(2 \cdot 2.7 + \frac{30.0 \cdot 2 \cdot 4 \cdot 160.2}{1000.0}\right)}{(35.4 + 30.0)}\right) \cdot 1000.0$$

$$= 670.1\ kN$$

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

$$= 2 \cdot 4 \cdot 160.2$$

$$= 1281.8\ kN$$

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

$$= min\ (380.6, 670.1, 1281.8)$$

$$= 380.6\ kN$$

Applied member tying load, $T_{SI,Ed} = 175.0\ kN$

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

Tying resistance-plate and bolts (373). Reference Pg 26 (continued)

$= \frac{175.0}{380.6}$

$= 0.459801$

$380.6\ kN \geq 175.0\ kN$ **(OK)**

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

Tying resistance-supported beam web (372). Reference Pg 27

Ultimate tensile strength of the supported beam, $f_{u,b1} = 410.0\ MPa$

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

Height of the end plate, $h_p = 290.0\ mm$

$\gamma_{Mu} = 1.1$

$$Tying\ capacity = \frac{\left(\frac{t_{w,b1} \cdot h_p \cdot f_{u,b1}}{\gamma_{Mu}}\right)}{1000.0}$$

$$= \frac{\left(\frac{9.5 \cdot 290.0 \cdot 410.0}{1.1}\right)}{1000.0}$$

$= 1026.9\ kN$

Applied member tying load, $T_{SI,Ed} = 175.0\ kN$

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

$= \frac{175.0}{1026.9}$

$= 0.170416$

$1026.9\ kN \geq 175.0\ kN$ **(OK)**

$0.170 \leq 1$ **(OK)**

Tying resistance-supporting column web (174). Reference Pg 29

Bolt row spacing, $p_1 = 70\ mm$

Bolt rows, $n_1 = 4$

Gage, $g = 90.0\ mm$

Column ultimate yield stress, $f_{u,2} = 410.0\ MPa$

Column k distance, $k_{col} = 36.9\ mm$

Column web thickness, $t_{w,col} = 13.8\ mm$

Column depth, $d_c = 320.5\ mm$

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

Distance between top and bottom hole on clip angle, $c = p_{1,total}$

$= 210.0\ mm$

Diameter of the hole, $d_0 = 22.0\ mm$

Depth of column between fillets, $d_2 = d_c - 2 \cdot k_{col}$

Tying resistance-supporting column web (174). Reference Pg 29 (continued)

$$= 320.5 - 2 \cdot 36.9$$
$$= 246.7\ mm$$

$$\eta_1 = \frac{\left(c - \frac{n_1 \cdot d_0}{2}\right)}{d_2}$$
$$= \frac{\left(210.0 - \frac{4 \cdot 22.0}{2}\right)}{246.7}$$
$$= 0.672882$$

$$\beta_1 = \frac{g}{d_2}$$
$$= \frac{90.0}{246.7}$$
$$= 0.364816$$

$$\gamma_1 = \frac{d_0}{d_2}$$
$$= \frac{22.0}{246.7}$$
$$= 0.0891771$$

$$\gamma_{Mu} = 1.1$$

$$M_{pl,Rd,u} = \frac{\left(\frac{f_{u,2} \cdot t_{w,col}^2}{4 \cdot \gamma_{Mu}}\right)}{1000.0}$$
$$= \frac{\left(\frac{410.0 \cdot 13.8^2}{4 \cdot 1.1}\right)}{1000.0}$$
$$= 17.7\ kN$$

$$Tying\ capacity = \left(\frac{8 \cdot M_{pl,Rd,u}}{(1 - \beta_1)}\right) \cdot \left(\eta_1 + 1.5 \cdot \sqrt{1 - \beta_1} \cdot \sqrt{1 - \gamma_1}\right)$$
$$= \left(\frac{8 \cdot 17.7}{(1 - 0.364816)}\right) \cdot \left(0.672882 + 1.5 \cdot \sqrt{1 - 0.364816} \cdot \sqrt{1 - 0.0891771}\right)$$
$$= 405.4\ kN$$

Applied member tying load, $T_{SI,Ed} = 175.0\ kN$

$$Unity = \frac{T_{SI,Ed}}{Tying\ capacity}$$
$$= \frac{175.0}{405.4}$$
$$= 0.431673$$

$405.4\ kN \geq 175.0\ kN$ **(OK)**

Tying resistance-supporting column web (174). Reference Pg 29 (continued)

$0.432 \leq 1$ **(OK)**

Rupture of web weld (455). Reference 4.5.3.3, Part 1-8

Member slope, $\alpha = 0.0\ °$

Correlation factor for fillet welds, $\beta_w = 0.85$

Nominal ultimate tensile strength of the weaker part joined, $f_{u,weaker} = 410.0\ MPa$

Tensile strength, $f_u = 410.0\ MPa$

Weld size to web, $w_w = 6.0\ mm$

Plate depth, $h_p = 290.0\ mm$

Web thickness, $t_w = 9.5\ mm$

k distance, $k = 26.2\ mm$

Full section depth, $h = 412.8\ mm$

Angle between longitudinal axis of weld and member axis, $\theta = 90.0 - |\alpha|$

$= 90.0 - |0.0|$

$= 90.0\ °$

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

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

$$\text{Effective weld size, } s_e = min\left(w_w, \frac{\left(\frac{f_u \cdot t_w}{\sqrt{3} \cdot \gamma_{M2,1\text{-}1}}\right)}{2 \cdot 0.707 \cdot \left(\frac{\left(\frac{f_{u,weaker}}{\sqrt{3}}\right)}{\beta_w \cdot \gamma_{M2,1\text{-}8}}\right)}\right)$$

$$= min\left(6.0, \frac{\left(\frac{410.0 \cdot 9.5}{\sqrt{3} \cdot 1.1}\right)}{2 \cdot 0.707 \cdot \left(\frac{\left(\frac{410.0}{\sqrt{3}}\right)}{0.85 \cdot 1.25}\right)}\right)$$

$= 6.0\ mm$

$\gamma_{Mu} = 1.1$

Rupture of web weld (455). Reference 4.5.3.3, Part 1-8 (continued)

$$\text{Weld strength per unit length, } F_r = \frac{18.0 \cdot s_e \cdot \left(\frac{\left(\frac{f_{u,weaker}}{\sqrt{3}}\right)}{\beta_w \cdot \gamma_{Mu}}\right) \cdot 1}{1000.0}$$

$$= \frac{18.0 \cdot 6.0 \cdot \left(\frac{\left(\frac{410.0}{\sqrt{3}}\right)}{0.85 \cdot 1.1}\right) \cdot 1}{1000.0}$$

$$= 27.3\ kN$$

$$\text{Weld length, } l_w = min\ (h - 2 \cdot k, h_p - 2 \cdot w_w)$$

$$= min\ (412.8 - 2 \cdot 26.2, 290.0 - 2 \cdot 6.0)$$

$$= 278.0\ mm$$

$$Tying\ capacity = 0.1 \cdot F_r \cdot l_w$$

$$= 0.1 \cdot 27.3 \cdot 278.0$$

$$= 597.1\ kN$$

$$\text{Applied member tying load, } T_{SI,Ed} = 175.0\ kN$$

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

$$= \frac{175.0}{597.1}$$

$$= 0.293083$$

$597.1\ kN \geq 175.0\ kN$ **(OK)**

$0.293 \leq 1$ **(OK)**

Results summary

End Plate on right end of Beam B_14 [15]

Design calculation summary for member [15], right end

Desc. Ref.:	Calc. Num.	Unity Ratio	Resistance	Design Force	Green book ref.
4:	(8)	0.025	393.7 kN	10.0 kN	Pg 17
8(i):	(1)	0.017	601.6 kN	10.0 kN	Pg 21
8(ii):	(110)	0.017	601.6 kN	10.0 kN	Pg 22
8(iii):	(110)	0.017	601.6 kN	10.0 kN	Pg 22
9(i):	(15)	0.014	725.1 kN	10.0 kN	Pg 23
9(ii):	(21)	0.012	869.4 kN	10.0 kN	Pg 23
9(iii):	(252)	0.014	691.0 kN	10.0 kN	Pg 23
10:	(409)	0.007	1499.1 kN	10.0 kN	Pg 24
11:	(373)	0.460	380.6 kN	175.0 kN	Pg 26
12:	(372)	0.170	1026.9 kN	175.0 kN	Pg 27
14:	(174)	0.432	405.4 kN	175.0 kN	Pg 29

Check number reference

4:	Supported beam-web shear
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
11:	Tying resistance-plate and bolts (SI)
12:	Tying resistance-supported beam web (SI)
14:	Tying resistance-supporting column web (SI)

Connection strength

	Unity ratio:
Shear:	0.025
Tying resistance:	0.460

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 2 - Partial depth end plate - Tying resistance, page 44
P358 Simple Joints.pdf