SDS2 Steel Connection Design: Connection Cube Report

Cube: Full depth end plate
Revision: 0
Project: 0954_Validation_Eurocode_3UK
Engineer:
Fabricator: Validation_Eurocode_3UK

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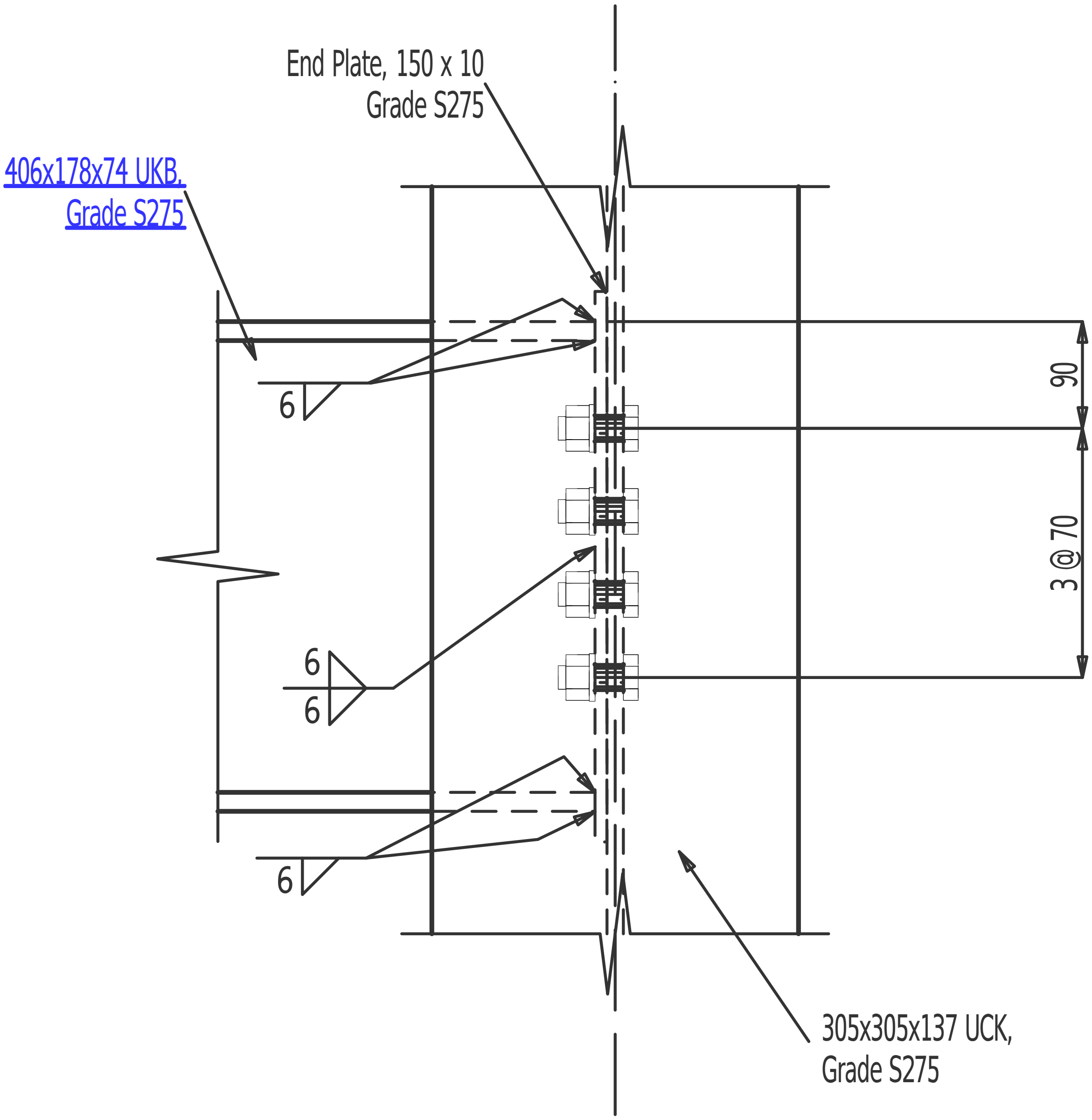
Full depth end plate [2] at X=5000, Y=0 Elev=2294



ELEVATION VIEW

Beam B_2 [2]

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, Extended end plate
	Welded extended tee: No
	Safety Automatic
	Expand Vertical bolt spacing: Auto
	Use web extension: If Required
Elevation:	2500
Minus Dim:	7.0 mm (AUTO)
Mtrl Setback:	17.0 mm (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	400.0 kN
Moment:	0.0 kN·m (AUTO)
Tension:	0.0 kN
Compress:	0.0 kN
Tying:	350.0 kN (AUTO)

B_2 [2] Connection strength check: RIGHT END

Member end summary

Connecting nodes

Node 1

Column:	C_9 [1]
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*:**	400.0 kN
***Tying force*:**	350.0 kN
***Tying force in a flange*:**	108.0 kN
***Tying force in the web*:**	134.0 kN

Design load notes

- Reaction has been input
- Design reaction is 60.6 % of the allowable uniform steel beam load.

Connection summary

- END PLATE SHEAR CONNECTION, EXTENDED TO BEAM FLANGES

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:	464.0 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
Flange weld:	**Weld size to flange, w_f:**	6.0 mm
	Weld type:	Double fillet
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:	115.4 mm
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit:	7
(at dd) Not Locked:	147

Expanded design calculation

Supported beam-web shear (2). Reference Pg 86

Web yield stress, $f_y = 275.0\ MPa$

Bottom flange thickness, $t_{f,b} = 16.0\ mm$

Top flange thickness, $t_{f,t} = 16.0\ mm$

Web thickness, $t_w = 9.5\ mm$

Full section depth, $h = 412.8\ mm$

Applied member shear, $V_{Ed} = 400.0\ kN$

$\gamma_{M0} = 1$

Cross-sectional shear area, $A_v = 9450.9\ mm^2$

Flange width, $b = 179.5\ mm$

Flange thickness, $t_f = 16.0\ mm$

Root radius, $r_b = 10.2\ mm$

$\eta = 1$

Web depth, $h_w = h - t_{f,t} - t_{f,b}$

$= 412.8 - 16.0 - 16.0$

$= 380.8\ mm$

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

$= max\left(9450.9 - 2 \cdot 179.5 \cdot 16.0 + \left(9.5 + 2 \cdot 10.2\right) \cdot 16.0, 1 \cdot 380.8 \cdot 9.5\right)$

$= 4185.3\ mm^2$

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

$$= \left(\frac{400.0}{\left(\frac{275.0 \cdot 4185.3}{\sqrt{3} \cdot 1}\right)}\right) \cdot 1000.0$$

$= 0.601951$

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

$$= \frac{\left(\frac{275.0 \cdot 4185.3}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$= 664.5\ kN$

$664.5\ kN \geq 400.0\ kN$ **(OK)**

$0.602 \leq 1$ **(OK)**

Supported beam-welds (24). Reference Pg 85

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 87

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, $V_{Ed} = 400.0\ kN$

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

$= \frac{400.0}{601.6}$

$= 0.664894$

$601.6\ kN \geq 400.0\ kN$ **(OK)**

$0.665 \leq 1$ **(OK)**

Connection-bearing on plate (110). Reference Pg 87

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, $p_1 = 70\ mm$

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, $p_1 = 70.0\ mm$

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

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

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

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

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

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

$$= 112.6\ kN$$

End bolt resistance

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

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

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

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

$$= \frac{\left(\frac{2.11818 \cdot 1 \cdot 410.0 \cdot 20.0 \cdot 10.0}{1.25}\right)}{1000.0}$$

$$= 139.0\ kN$$

Bolt shear resistance

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

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

$(0.8 \cdot F_{v,Rd} \cdot \beta_{LF} = 0.8 \cdot 94.0 \cdot 1 = 75.2\ kN) < (F_{b,Rd,e} = 139.0\ kN)$

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

Fastener resistance limited by bolt shear resistance

Bearing resistance, $F_{b,Rd} = min\left(F_{b,Rd,e},F_{b,Rd,i},0.8 \cdot F_{v,Rd} \cdot \beta_{LF}\right)$
$$= min\left(139.0,112.6,0.8 \cdot 94.0 \cdot 1\right)$$
$$= 75.2\ kN$$

$Shear\ capacity = F_{b,Rd} \cdot n_1 \cdot n_2$
$$= 75.2 \cdot 4 \cdot 2$$
$$= 601.6\ kN$$

Applied member shear, $V_{Ed} = 400.0\ kN$

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$
$$= \frac{400.0}{601.6}$$
$$= 0.664894$$

$601.6\ kN \geq 400.0\ kN$ **(OK)**

$0.665 \leq 1$ **(OK)**

Connection-bearing on support (110). Reference Pg 87

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, $p_1 = 70.0\ mm$

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

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

$$k_1 = min\left(\frac{1.4 \cdot p_2}{d_0} - 1.7, 2.5\right) = min\left(\frac{1.4 \cdot 90.0}{22.0} - 1.7, 2.5\right) = 2.5$$

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

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

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

$= 183.5\ kN$

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

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

$$= 226.3\ kN$$

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

$$= min\left(226.3,183.5,0.8 \cdot 94.0 \cdot 1\right) = 75.2\ kN$$

$$Shear\ capacity = F_{b,Rd} \cdot n_1 \cdot n_2 = 75.2 \cdot 4 \cdot 2 = 601.6\ kN$$

Applied member shear, $V_{Ed} = 400.0\ kN$

$$Unity = \frac{V_{Ed}}{Shear\ capacity} = \frac{400.0}{601.6} = 0.664894$$

$601.6\ kN \geq 400.0\ kN$ **(OK)**

$0.665 \leq 1$ **(OK)**

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

Not necessary to check in-plane bending.

$Unity = 0$

Supporting member-shear (409). Reference Pg 88

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, $d_0 = 22.0\ mm$

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

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

$= 1499.1\ kN$

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

$= 1499.1\ kN$

Supporting member-shear (409). Reference Pg 88 (continued)

Applied member shear, $V_{Ed} = 400.0\ kN$

$$Unity = \frac{V_{Ed}}{Shear\ capacity}$$
$$= \frac{400.0}{1499.1}$$
$$= 0.266827$$

$1499.1\ kN \geq 400.0\ kN$ **(OK)**

$0.267 \leq 1$ **(OK)**

Tying resistance-plate and bolts (373). Reference Pg 90

Size of the weld on the beam flange, $s_f = 6.0\ mm$

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

Distance from top of the beam to the first bolt row, $p = 90.0\ mm$

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, $t_{w,b1} = 9.5\ mm$

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

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

Bolt row, $n_1 = 4$

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

$$= \frac{(150.0 - 90.0)}{2}$$
$$= 30.0\ mm$$

$$e_{min} = e_2$$
$$= 30.0\ mm$$

$$m = \frac{(p_3 - t_{w,b1} - 2 \cdot 0.8 \cdot s_w)}{2}$$
$$= \frac{(90.0 - 9.5 - 2 \cdot 0.8 \cdot 6.0)}{2}$$
$$= 35.4\ mm$$

$$n = min\ (e_{min}, 1.25 \cdot m)$$
$$= min\ (30.0, 1.25 \cdot 35.4)$$
$$= 30.0\ mm$$

$$m_1 = p - t_{f,b1} - 0.8 \cdot s_f$$
$$= 90.0 - 16.0 - 0.8 \cdot 6.0$$
$$= 69.2\ mm$$

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

$$m_2 = h_{b1} - p - p_{1,total} - t_{f,b1} - 0.8 \cdot s_f$$

Tying resistance-plate and bolts (373). Reference Pg 90 (continued)

$= 412.8 - 90.0 - 210.0 - 16.0 - 0.8 \cdot 6.0$

$= 92.0\ mm$

$\alpha_t = 5.07117$

$\alpha_b = 5.05783$

$l_{eff,t} = max\left(\alpha_t \cdot m - \frac{(4 \cdot m + 1.25 \cdot e_2)}{2}, \frac{(4 \cdot m + 1.25 \cdot e_2)}{2}\right)$

$= max\left(5.07117 \cdot 35.4 - \frac{(4 \cdot 35.4 + 1.25 \cdot 30.0)}{2}, \frac{(4 \cdot 35.4 + 1.25 \cdot 30.0)}{2}\right)$

$= 90.1\ mm$

$l_{eff,b} = max\left(\alpha_b \cdot m - \frac{(4 \cdot m + 1.25 \cdot e_2)}{2}, \frac{(4 \cdot m + 1.25 \cdot e_2)}{2}\right)$

$= max\left(5.05783 \cdot 35.4 - \frac{(4 \cdot 35.4 + 1.25 \cdot 30.0)}{2}, \frac{(4 \cdot 35.4 + 1.25 \cdot 30.0)}{2}\right)$

$= 89.7\ mm$

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

Effective length, $l_{eff} = l_{eff,t} + p_{1,total} + l_{eff,b}$

$= 90.1 + 210.0 + 89.7$

$= 389.8\ mm$

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

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

$= \frac{33.0}{4}$

$= 8.2\ mm$

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

$k_2 = 0.9$

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

$\gamma_{Mu} = 1.1$

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

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

$= 160.2\ kN$

$\gamma_{Mu} = 1.1$

Tying resistance-plate and bolts (373). Reference Pg 90 (continued)

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

$$= \frac{\left(\frac{0.25 \cdot 389.8 \cdot 10.0^2 \cdot 410.0}{1.1}\right)}{1000000.0}$$

$$= 3.6\ kN \cdot m$$

$$M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$$
$$= 3.6\ kN \cdot m$$

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

$$= \left(\frac{(8 \cdot 30.0 - 2 \cdot 8.2) \cdot 3.6}{(2 \cdot 35.4 \cdot 30.0 - 8.2 \cdot (35.4 + 30.0))}\right) \cdot 1000.0$$
$$= 511.5\ kN$$

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

$$= \left(\frac{\left(2 \cdot 3.6 + \frac{30.0 \cdot 2 \cdot 4 \cdot 160.2}{1000.0}\right)}{(35.4 + 30.0)}\right) \cdot 1000.0$$

$$= 698.5\ kN$$

$$F_{Rd,u,3} = 2 \cdot n_1 \cdot F_{t,Rd,u}$$
$$= 2 \cdot 4 \cdot 160.2$$
$$= 1281.8\ kN$$

$$Tying\ capacity = min\ (F_{Rd,u,1}, F_{Rd,u,2}, F_{Rd,u,3})$$
$$= min\ (511.5, 698.5, 1281.8)$$
$$= 511.5\ kN$$

Applied member tying load, $T_{SI,Ed} = 350.0\ kN$

$$Unity = \frac{T_{SI,Ed}}{Tying\ capacity}$$
$$= \frac{350.0}{511.5}$$
$$= 0.684263$$

$511.5\ kN \geq 350.0\ kN$ **(OK)**

$0.684 \leq 1$ **(OK)**

Tying resistance-supported beam web (372). Reference Pg 92

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

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

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

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

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

$\gamma_{Mu} = 1.1$

$$Tying\ capacity = \frac{\left(\frac{t_{w,b1} \cdot \left(h_{b1} - t_{f,t,b1} - t_{f,b,b1}\right) \cdot f_{u,b1}}{\gamma_{Mu}}\right)}{1000.0}$$

$$= \frac{\left(\frac{9.5 \cdot (412.8 - 16.0 - 16.0) \cdot 410.0}{1.1}\right)}{1000.0}$$

$$= 1348.4\ kN$$

Applied member tying load, $T_{SI,Ed} = 350.0\ kN$

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

$$= \frac{350.0}{1348.4}$$

$$= 0.259567$$

$1348.4\ kN \geq 350.0\ kN$ **(OK)**

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

Tying resistance-flange welds (375). Reference Pg 93

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

Ultimate tensile strength of the weaker part, $f_u = 410.0\ MPa$

Weld length, $l_w = 150.0\ mm$

Size of the flange weld, $s = 6.0\ mm$

Weld throat, $a = 0.7 \cdot s$

$$= 0.7 \cdot 6.0$$

$$= 4.2\ mm$$

$\gamma_{Mu} = 1.1$

$$f_{vwd} = \frac{\left(\frac{f_u}{\sqrt{3}}\right)}{\beta_w \cdot \gamma_{Mu}}$$

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

Tying resistance-flange welds (375). Reference Pg 93 (continued)

$= 253.2\ MPa$

Weld strength, $F_{Rd,flangeweld} = \dfrac{2 \cdot f_{vwd} \cdot a \cdot (l_w - 2 \cdot s)}{1000.0}$

$= \dfrac{2 \cdot 253.2 \cdot 4.2 \cdot (150.0 - 2 \cdot 6.0)}{1000.0}$

$= 293.5\ kN$

$Tying\ capacity = 2 \cdot F_{Rd,flangeweld}$

$= 2 \cdot 293.5$

$= 586.9\ kN$

Applied member tying load, $T_{SI,Ed} = 350.0\ kN$

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

$= \dfrac{350.0}{586.9}$

$= 0.596354$

$586.9\ kN \geq 350.0\ kN$ **(OK)**

$0.596 \leq 1$ **(OK)**

Tying resistance-supporting column web (174). Reference Pg 94

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, $d_2 = d_c - 2 \cdot k_{col}$

$= 320.5 - 2 \cdot 36.9$

$= 246.7\ mm$

$$\eta_1 = \frac{\left(c - \dfrac{n_1 \cdot d_0}{2}\right)}{d_2}$$

$$= \frac{\left(210.0 - \dfrac{4 \cdot 22.0}{2}\right)}{246.7}$$

$= 0.672882$

Tying resistance-supporting column web (174). Reference Pg 94 (continued)

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

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

$$\gamma_{Mu} = 1.1$$

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

$$= \frac{\left(\frac{410.0 \cdot 13.8^2}{4 \cdot 1.1}\right)}{1000.0}$$

$$= 17.7\ kN$$

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

Applied member tying load, $T_{SI,Ed} = 350.0\ kN$

$$Unity = \frac{T_{SI,Ed}}{Tying\ capacity}$$
$$= \frac{350.0}{405.4}$$
$$= 0.863346$$

$405.4\ kN \geq 350.0\ kN$ **(OK)**

$0.863 \leq 1$ **(OK)**

Results summary

Extended End Plate on right end of Beam B_2 [2]

Design calculation summary for member [2], right end

Desc. Ref.:	Calc. Num.	Unity Ratio	Resistance	Design Force	Green book ref.
4:	(2)	0.602	664.5 kN	400.0 kN	Pg 86
8(i):	(1)	0.665	601.6 kN	400.0 kN	Pg 87
8(ii):	(110)	0.665	601.6 kN	400.0 kN	Pg 87
8(iii):	(110)	0.665	601.6 kN	400.0 kN	Pg 87
10:	(409)	0.267	1499.1 kN	400.0 kN	Pg 88
11:	(373)	0.684	511.5 kN	350.0 kN	Pg 90
12:	(372)	0.260	1348.4 kN	350.0 kN	Pg 92
13:	(375)	0.596	586.9 kN	350.0 kN	Pg 93
14:	(174)	0.863	405.4 kN	350.0 kN	Pg 94

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
10:	Supporting member-shear
11:	Tying resistance-plate and bolts (SI)
12:	Tying resistance-supported beam web (SI)
13:	Tying resistance-flange welds (SI)
14:	Tying resistance-supporting column web (SI)

Warnings

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

Notes and conclusions

- Effective web weld length = Beam d distance.
- Effective flange weld length = Beam flange width.
- CONNECTION DESIGN FAILURE
  - Moment/extended end plate width less than beam flange width ⚠
- Weld size satisfies basic requirements of Check 2 from the Green Book.

Additional notes

Example 1 - Full depth end plate, page 97, and
Example 2 - Full depth end plate Tying resistance, page 102,
P358 Simple Joints.pdf