SDS2 Steel Connection Design: Connection Cube Report

Cube: Moment end plate
Revision: 0
Project: Validation_MEP_Eurocode_3UK
Engineer:
Fabricator: Validation_Eurocode_3UK

Generated by SDS2 x 2025.01 on Friday, Oct 11, 2024

Schedule of Minimums -- Structural Members

Nom. Depth	Bolt Dia.	Num. Rows	TOS to First Hole
254	20	2	75
305	20	3	75
356	20	4	90
406	20	4	90
457	20	5	90
533	20	6	90
610	20	7	90
686	20	8	90
762	20	9	90
838	20	10	90
914	20	11	90

Moment end plate [5] at X=4705, Y=-10000 Elev=2233

Project settings

Design method

Connection design method: EURO05UK

Design method for moment connections: Elastic

Always provide stabilizer plates: No

Use alternate eccentricity for extended shear tabs: No

Always base shear tab on flexible support condition: No

Beam design loads

Design reactions are 50% of the uniform allowable load, UNO

Base beam design moment: 85% maximum allowable moment

Seismic moment connections: No

Use inner flange plates for beam splice moment: No

Auto-calculate tying loads: No

Seismic moment frame type: OMF

Connection material specifications

Tee sections: S275

Angle sections: S275

Channel sections: S275

Base/Cap plate schedule: S275

Beam composite design

Non-composite design, UNO

When composite used, default settings:

% Uniform composite load: 50

Concrete strength: 4.35113 MPa

Concrete thickness: 4 mm

Braces

% Allowable tension load for horizontal braces: 50

% Allowable compression load for horizontal braces: 50

% Allowable tension load for vertical braces: 50

% Allowable compression load for vertical braces: 50

% Allowable tension load for intersection horizontal braces: 20

% Allowable compression load for intersection horizontal braces: 20

% Allowable tension load for intersection vertical braces: 20

% Allowable compression load for intersection vertical braces: 20

Add gusset-beam interface forces to beam: No

UFM special case design: No special case

Wide flange vertical brace to gusset web connection: Web plates

Wide flange vertical brace to gusset flange connection: Flange claw angles

Seismic vertical brace gusset design: SCBF

Use AISC seismic design (1st ed.) gusset geometry for 2 point gusset: No

Weld design settings

Weld standard: AWS

Weld material tensile strength: 70.0532 MPa

Weld electrode type: E480xx

Bolt settings

AISC 13th slip critical bolt design: Design all bolts to prevent slip at the required strength level

Default non-moment bolt diameter: 20 mm

Default non-moment bolt type: 8.8A

Default material grades

Angle: S275

Channel: S275

Flat bar: S275

Grating: A-1011

HSS Tube: S235H

Pipe: S235H

Plate: S275

Rebar: 60

Round and square bar: S275

Turnbuckle / Clevis / Pin: C-1035

Wide flange: S275

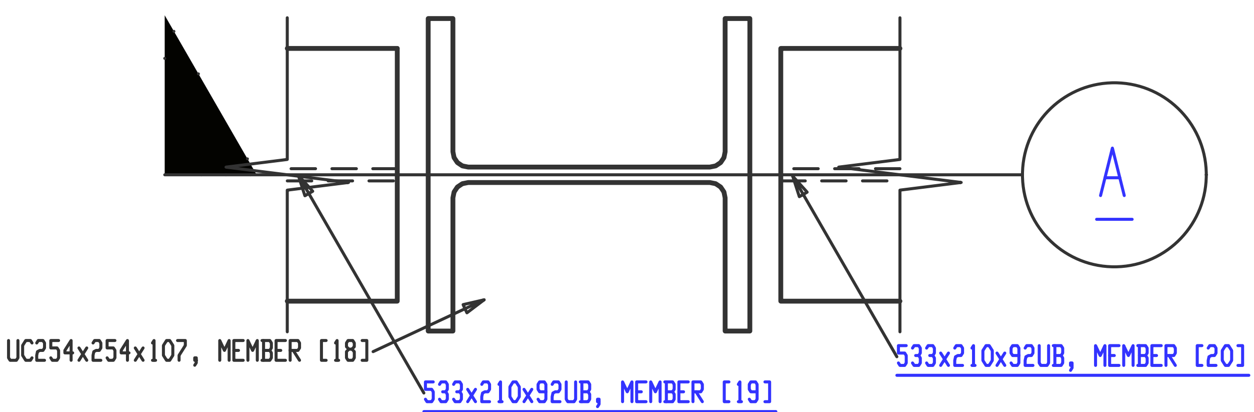
WT: S275

Concrete reinforcement defaults

Bar size: #4

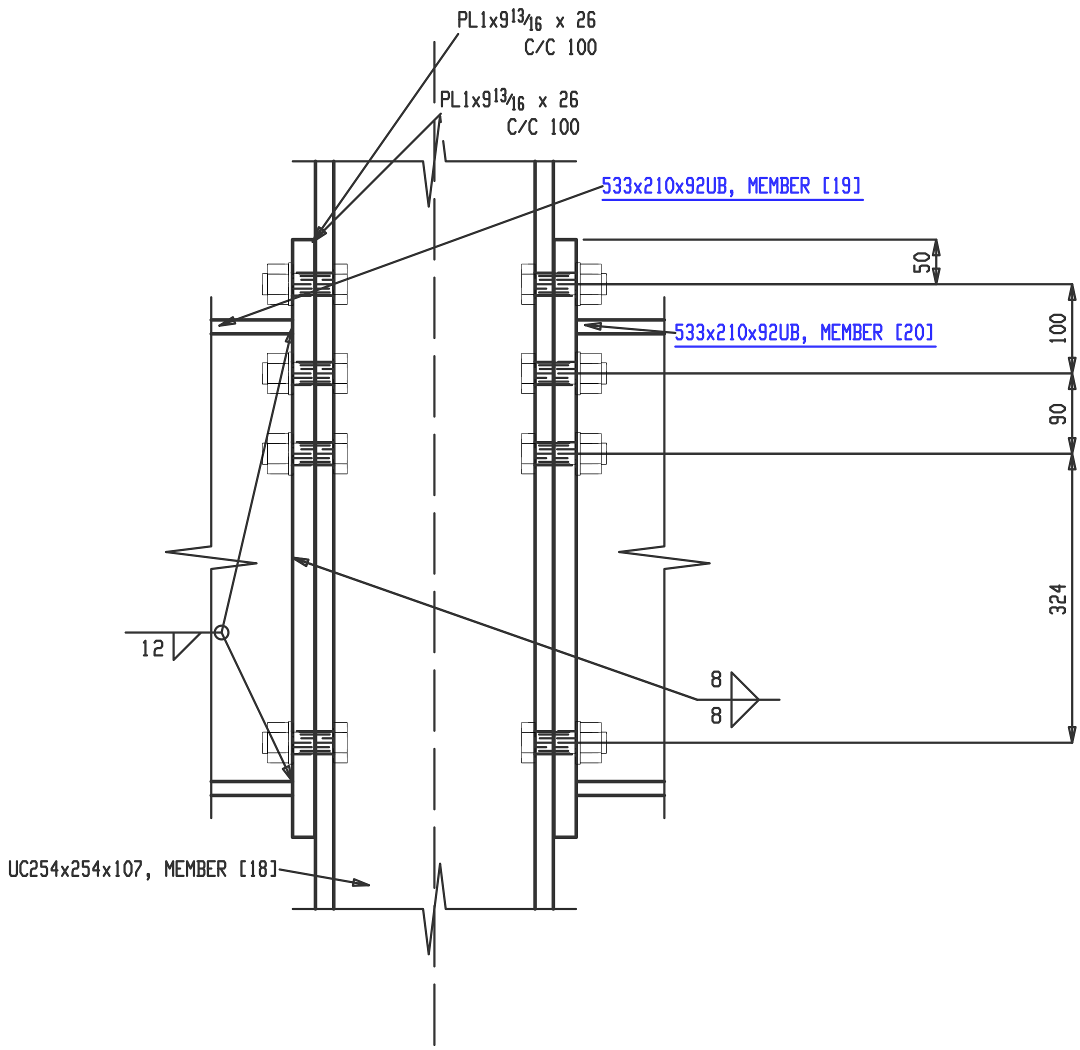
Bar material: A615

Bar grade: 60



TOP SIDE VIEW

Moment end plate



A
Section A
Moment end plate ELEVATION

Beam B_17 [19]

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:	4841
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

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: Never
	Minimum Setup: No, Bolted moment
	, Not designed for column flange stiffe
	ner, Design for column web doubler
Elevation:	2500
Minus Dim:	134.0 mm (AUTO)
Mtrl Setback:	159.0 mm (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	20.0 kN
Story shear:	0.0 kN
Moment:	-400.0 kN·m
Tension:	0.0 kN
Compress:	0.0 kN
Tying:	0.0 kN (AUTO)

B_17 [19] Connection strength check: RIGHT END

Member end summary

Connecting nodes

Node 1

Column:	C_16 [18]
Section size:	UC254x254x107
End 0 elevation:	0 mm
End 1 elevation:	5000 mm
Framing condition:	Flange of Column
Material grade:	S275a
Supporting member thickness, t_2:	20.5 mm

Factored loads

***Shear*:**	20.0 kN
***Moment*:**	-400.0 kN·m

Design load notes

- Reaction has been input
- Moment has been input
- Design reaction is 1.9 % of the allowable uniform steel beam load.
- Design moment is 61.6 % of Mc,Rd.

Connection summary

Connection details

Connection summary:	EXTENDED MOMENT END PLATE	
		(Tension bolts top only)
Plate:	**Grade:**	S275
	Tensile strength, f_u:	410.0 MPa
	Yield stress, f_y:	265.0 MPa

Connection details (continued)

	Thickness, t:	25.0 mm
	Depth, h:	670.0 mm
	Width, b:	250.0 mm
Welds:	**Weld metal strength, F_{exx}:**	0.0 MPa
Weld to web:	**Weld type:**	Double fillet
	Weld leg size, s:	8.0 mm
Weld to flange:	**Weld type:**	Single fillet
	Weld leg size, s:	12.0 mm
Bolts:	**Bolt type:**	8.8, Category A
	Hole type in connection:	Standard round
	Bolt diameter, d:	24
	Bolt rows, n_1:	4
	Distance from flange to first bolt, x:	40.0 mm
	Bolt gage, p_3:	100.0 mm
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit:	1
(at dd) Not Locked:	804

Beam B_18 [20]

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:	4841
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
Plastic section modulus about the major axis, $W_{pl,y}$:	2360087.4 mm^3

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: Never
	Minimum Setup: No, Bolted moment
	, Not designed for column flange stiffe
	ner, Design for column web doubler
Elevation:	2500
Minus Dim:	134.0 mm (AUTO)
Mtrl Setback:	159.0 mm (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	20.0 kN
Story shear:	0.0 kN
Moment:	400.0 kN·m
Tension:	0.0 kN
Compress:	0.0 kN
Tying:	0.0 kN (AUTO)

B_18 [20] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_16 [18]
Section size:	UC254x254x107
End 0 elevation:	0 mm
End 1 elevation:	5000 mm
Framing condition:	Flange of Column
Material grade:	S275a
Supporting member thickness, t_2:	20.5 mm

Factored loads

***Shear*:**	20.0 kN
***Moment*:**	400.0 kN·m

Design load notes

- Reaction has been input
- Moment has been input
- Design reaction is 1.9 % of the allowable uniform steel beam load.
- Design moment is 61.6 % of Mc,Rd.

Connection summary

Connection details

Connection summary:	EXTENDED MOMENT END PLATE	
		(Tension bolts top only)
Plate:	**Grade:**	S275
	Tensile strength, f_u:	410.0 MPa
	Yield stress, f_y:	265.0 MPa

Connection details (continued)

	Thickness, t:	25.0 mm
	Depth, h:	670.0 mm
	Width, b:	250.0 mm
Welds:	**Weld metal strength, F_{exx}:**	0.0 MPa
Weld to web:	**Weld type:**	Double fillet
	Weld leg size, s:	8.0 mm
Weld to flange:	**Weld type:**	Single fillet
	Weld leg size, s:	12.0 mm
Bolts:	**Bolt type:**	8.8, Category A
	Hole type in connection:	Standard round
	Bolt diameter, d:	24
	Bolt rows, n_1:	4
	Distance from flange to first bolt, x:	40.0 mm
	Bolt gage, p_3:	100.0 mm
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit:	1
(at dd) Not Locked:	804

Expanded design calculation

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

Bolt diameter, $d = 24.0\ mm$

Interior row edge distance top, $p_{ft} = 60.0\ mm$

Bolt row spacing, $p_1 = 90.0\ mm$

Interior bolt rows top, $n_{i,top} = 2$

Yield stress, $f_y = 275.0\ MPa$

Projected flange thickness, $t_f = 15.6\ mm$

Web thickness, $t_w = 10.1\ mm$

Full section depth, $h = 533.1\ mm$

Applied member shear, $V_{Ed} = 20.0\ kN$

Length of the web effective shear region, $l_{eff} = min\left(h - \left(p_{ft} + \left(n_{i,top} - 1\right) \cdot p_1 + 2 \cdot d\right) - t_f, \frac{\left(h - 2 \cdot t_f\right)}{2}\right)$

$= min\left(533.1 - \left(60.0 + \left(2 - 1\right) \cdot 90.0 + 2 \cdot 24.0\right) - 15.6, \frac{\left(533.1 - 2 \cdot 15.6\right)}{2}\right)$

$= 250.9\ mm$

Web shear area, $A_{v,w} = l_{eff} \cdot t_w$

$= 250.9 \cdot 10.1$

$= 2534.6\ mm^2$

$\gamma_{M0} = 1$

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

$$= \left(\frac{20.0}{\left(\frac{275.0 \cdot 2534.6}{\sqrt{3} \cdot 1}\right)}\right) \cdot 1000.0$$

$= 0.0496993$

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

$$= \frac{\left(\frac{275.0 \cdot 2534.6}{\sqrt{3} \cdot 1}\right)}{1000.0}$$

$= 402.4\ kN$

$402.4\ kN \geq 20.0\ kN$ **(OK)**

$0.050 \leq 1$ **(OK)**

Supported beam-moment strength (59). Reference 6.2, Part 1-1

Root radius, $r = 12.7\ mm$

Yield stress, $f_y = 275.0\ MPa$

Flange thickness, $t_f = 15.6\ mm$

Beam section overall breadth, $b = 209.3\ mm$

Web thickness, $t_w = 10.1\ mm$

Beam section overall depth, $h = 533.1\ mm$

Plastic section modulus about the major axis, $W_{pl,y} = 2360087.4\ mm^3$

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

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

Applied member shear, $V_{Ed} = 20.0\ kN$

Applied member moment, $M_{Ed} = 400.0\ kN \cdot m$

Design axial force, $N_{Ed} = max\ (N_{t,Ed}, N_{c,Ed})$

$= max\ (0.0, 0.0)$

$= 0.0\ kN$

Cross-sectional shear area, $A_v = 11737.8\ mm^2$

Flange width, $b = 209.3\ mm$

Flange thickness, $t_f = 15.6\ mm$

Root radius, $r_b = 12.7\ mm$

$\eta = 1$

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

$= 533.1 - 15.6 - 15.6$

$= 501.9\ mm$

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

$= max\ (11737.8 - 2 \cdot 209.3 \cdot 15.6 + (10.1 + 2 \cdot 12.7) \cdot 15.6, 1 \cdot 501.9 \cdot 10.1)$

$= 5761.4\ mm^2$

$\gamma_{M0} = 1$

Design plastic shear resistance, $V_{pl,Rd} = \dfrac{\left(\dfrac{f_y \cdot A_{v,w}}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{275.0 \cdot 5761.4}{\sqrt{3} \cdot 1}\right)}{1000.0}$

$= 914.8\ kN$

Gross section area, $A_g = b \cdot t_f \cdot 2 + (h - 2 \cdot t_f) \cdot t_w + 4 \cdot r^2 \cdot \left(1 - \dfrac{\pi}{4}\right)$

$= 209.3 \cdot 15.6 \cdot 2 + (533.1 - 2 \cdot 15.6) \cdot 10.1 + 4 \cdot 12.7^2 \cdot (1 - 0.785398)$

$= 11737.8\ mm^2$

$\gamma_{M0} = 1$

Supported beam-moment strength (59). Reference 6.2, Part 1-1 (continued)

Design plastic normal resistance, $N_{pl,Rd} = \frac{\left(\frac{f_y \cdot A_g}{\gamma_{M0}}\right)}{1000.0}$

$= \frac{\left(\frac{275.0 \cdot 11737.8}{1}\right)}{1000.0}$

$= 3227.9\ kN$

Reduced yield stress due to shear, $f_{y,r} = f_y$

$= 275.0\ MPa$

$\gamma_{M0} = 1$

Design plastic moment resistance, $M_{pl,Rd} = \frac{\left(\frac{f_{y,r} \cdot W_{pl,y}}{\gamma_{M0}}\right)}{1000000.0}$

$= \frac{\left(\frac{275.0 \cdot 2360087.4}{1}\right)}{1000000.0}$

$= 649.0\ kN \cdot m$

$Moment\ capacity = M_{pl,Rd}$

$= 649.0\ kN \cdot m$

$Unity = \frac{|M_{Ed}|}{Moment\ capacity}$

$= \frac{|400.0|}{649.0}$

$= 0.616333$

$649.0\ kN \cdot m \geq (|400.0| = 400.0\ kN \cdot m)$ **(OK)**

$0.616 \leq 1$ **(OK)**

Supported beam-welds (52). Reference Pg 16

Web thickness, $t_w = 10.1\ mm$

Minimum specified total throat thickness, $a_{min} = t_w$

$= 10.1\ mm$

Minimum specified weld leg size, $s_{min} = \frac{\left(\frac{a_{min}}{0.707}\right)}{2}$

$= \frac{\left(\frac{10.1}{0.707}\right)}{2}$

$= 7.1\ mm$

Supported beam-welds (52). Reference Pg 16 (continued)

$8.0\ mm \geq \left(s_{min} = 7.1\ mm\right)$ **(OK)**

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

Connection-bolt shear (49). Reference Pg 27

Bolt rows that carry tension, $n_{1,t} = 3$

Bolt rows, $n_1 = 4$

Bolt rows that are in shear only, $n_{1,s} = n_1 - n_{1,t}$

$= 4 - 3$

$= 1$

Tensile stress area of the bolt, $A = 352.5\ mm^2$

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

$0.0\ mm \leq \left(\frac{d}{3} = \frac{24.0}{3} = 8.0\ mm\right)$

Not necessary to reduce shear capacity for packing.

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

$$= \frac{\left(\frac{480.0 \cdot 352.5}{1.25}\right) \cdot 1}{1000.0}$$

$= 135.4\ kN$

$Shear\ capacity = F_{v,Rd} \cdot 2 \cdot n_{1,s} + 0.28 \cdot F_{v,Rd} \cdot 2 \cdot n_{1,t}$

$= 135.4 \cdot 2 \cdot 1 + 0.28 \cdot 135.4 \cdot 2 \cdot 3$

$= 498.1\ kN$

Applied member shear, $V_{Ed} = 20.0\ kN$

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

$$= \frac{20.0}{498.1}$$

$= 0.0401526$

$498.1\ kN \geq 20.0\ kN$ **(OK)**

$0.040 \leq 1$ **(OK)**

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

Tensile strength, $f_u = 410.0\ MPa$

Plate thickness, $t_p = 25.0\ mm$

Bolt column spacing, $p_3 = 100.0\ mm$

Column edge distance, $e_2 = 75.0\ mm$

Bolt row spacing, $p_1 = 90\ mm$

Bolt diameter, $d = 24.0\ mm$

Bolt columns, $n_2 = 2$

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

Total bolt rows, $n_{1,out} = 1$

Total bolt rows, $n_1 = 4$

Bolt row inside the beam, $n_{1,in} = n_1 - n_{1,out}$

$= 4 - 1$

$= 3$

Connection-bearing on plate, inside bolt (110)

Hole diameter, $d_0 = 26.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt row spacing, $p_1 = 90.0\ mm$

$$\alpha_d = \frac{p_1}{3 \cdot d_0} - 0.25$$
$$= \frac{90.0}{3 \cdot 26.0} - 0.25$$
$$= 0.903846$$

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$$
$$= min\left(0.903846, \frac{800.0}{410.0}, 1\right)$$
$$= 0.903846$$

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right)$$
$$= min\left(\frac{2.8 \cdot 75.0}{26.0} - 1.7, 2.5\right)$$
$$= 2.5$$

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

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

$$= \frac{\left(\frac{2.5 \cdot 0.903846 \cdot 410.0 \cdot 24.0 \cdot 25.0}{1.25}\right)}{1000.0}$$

$$= 444.7\ kN$$

End bolt resistance

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

End bolt resistance (continued)

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right)$$
$$= min\left(\frac{2.8 \cdot 75.0}{26.0} - 1.7, 2.5\right)$$
$$= 2.5$$

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

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

$$= \frac{\left(\frac{2.5 \cdot 1 \cdot 410.0 \cdot 24.0 \cdot 25.0}{1.25}\right)}{1000.0}$$
$$= 492.0\ kN$$

Bearing resistance on the plate, $F_{b,Rd,p} = min\left(F_{b,Rd,e}, F_{b,Rd,i}\right)$
$$= min\left(492.0, 444.7\right)$$
$$= 444.7\ kN$$

Bearing resistance of the bolts inside the beam, $F_{b,Rd,in} = F_{b,Rd,p} \cdot n_{1,in} \cdot n_2$
$$= 444.7 \cdot 3 \cdot 2$$
$$= 2668.2\ kN$$

Connection-bearing on plate, outside bolt (110)

Hole diameter, $d_0 = 26.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

No inner bolts exist in this load direction.

End bolt resistance

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

Bolt column spacing, $p_3 = 100.0\ mm$

Bolt column spacing, $p_3 = 100.0\ mm$

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, \frac{1.4 \cdot p_3}{d_0} - 1.7, 2.5\right)$$
$$= min\left(\frac{2.8 \cdot 75.0}{26.0} - 1.7, \frac{1.4 \cdot 100.0}{26.0} - 1.7, 2.5\right)$$
$$= 2.5$$

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

End bolt resistance (continued)

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

$= \dfrac{\left(\dfrac{2.5 \cdot 1 \cdot 410.0 \cdot 24.0 \cdot 25.0}{1.25}\right)}{1000.0}$

$= 492.0\ kN$

Bearing resistance on the plate, $F_{b,Rd,p} = min\ (F_{b,Rd,e}, F_{b,Rd,i})$

$= min\ (492.0, 0.0)$

$= 0.0\ kN$

Bearing resistance of the bolts outside the beam, $F_{b,Rd,in} = F_{b,Rd,p} \cdot n_{1,out} \cdot n_2$

$= 0.0 \cdot 1 \cdot 2$

$= 0.0\ kN$

Bearing resistance, $F_{b,Rd} = F_{b,Rd,in} + F_{b,Rd,in}$

$= 2668.2 + 0.0$

$= 2668.2\ kN$

$Shear\ capacity = F_{b,Rd}$

$= 2668.2\ kN$

Applied member shear, $V_{Ed} = 20.0\ kN$

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

$= \dfrac{20.0}{2668.2}$

$= 0.0074957$

$2668.2\ kN \geq 20.0\ kN$ **(OK)**

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

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

Tensile strength, $f_u = 410.0\ MPa$

Plate thickness, $t_p = 20.5\ mm$

Bolt column spacing, $p_3 = 100.0\ mm$

Column edge distance, $e_2 = 79.4\ mm$

Bolt row spacing, $p_1 = 90\ mm$

Row edge distance of the outside bolts, $e_1 = 253974.6\ mm$

Bolt diameter, $d = 24.0\ mm$

Bolt columns, $n_2 = 2$

Total bolt rows, $n_{1,out} = 1$

Total bolt rows, $n_1 = 4$

Bolt row inside the beam, $n_{1,in} = n_1 - n_{1,out}$

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

$= 4 - 1$

$= 3$

Connection-bearing on support, inside bolt (110)

Hole diameter, $d_0 = 26.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

Bolt row spacing, $p_1 = 90.0\ mm$

$$\alpha_d = \frac{p_1}{3 \cdot d_0} - 0.25$$
$$= \frac{90.0}{3 \cdot 26.0} - 0.25$$
$$= 0.903846$$

$$\alpha_b = min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1\right)$$
$$= min\left(0.903846, \frac{800.0}{410.0}, 1\right)$$
$$= 0.903846$$

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right)$$
$$= min\left(\frac{2.8 \cdot 79.4}{26.0} - 1.7, 2.5\right)$$
$$= 2.5$$

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

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

$$= \frac{\left(\frac{2.5 \cdot 0.903846 \cdot 410.0 \cdot 24.0 \cdot 20.5}{1.25}\right)}{1000.0}$$

$$= 364.6\ kN$$

End bolt resistance

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

$$k_1 = min\left(\frac{2.8 \cdot e_2}{d_0} - 1.7, 2.5\right)$$
$$= min\left(\frac{2.8 \cdot 79.4}{26.0} - 1.7, 2.5\right)$$
$$= 2.5$$

End bolt resistance (continued)

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

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

$= \dfrac{\left(\dfrac{2.5 \cdot 1 \cdot 410.0 \cdot 24.0 \cdot 20.5}{1.25}\right)}{1000.0}$

$= 403.4\ kN$

Bearing resistance on the supporting member, $F_{b,Rd,2} = F_{b,Rd,i}$

$= 364.6\ kN$

Bearing resistance of the bolts inside the beam, $F_{b,Rd,in} = F_{b,Rd,2} \cdot n_{1,in} \cdot n_2$

$= 364.6 \cdot 3 \cdot 2$

$= 2187.9\ kN$

Connection-bearing on support, outside bolt (110)

Hole diameter, $d_0 = 26.0\ mm$

$f_{ub} = 800.0\ MPa$

Inner bolt resistance

No inner bolts exist in this load direction.

End bolt resistance

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

$= \dfrac{253974.6}{3 \cdot 26.0}$

$= 3256.08$

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

$= min\left(3256.08, \dfrac{800.0}{410.0}, 1\right)$

$= 1$

Bolt column spacing, $p_3 = 100.0\ mm$

Bolt column spacing, $p_3 = 100.0\ mm$

$k_1 = min\left(\dfrac{2.8 \cdot e_2}{d_0} - 1.7, \dfrac{1.4 \cdot p_3}{d_0} - 1.7, 2.5\right)$

$= min\left(\dfrac{2.8 \cdot 79.4}{26.0} - 1.7, \dfrac{1.4 \cdot 100.0}{26.0} - 1.7, 2.5\right)$

$= 2.5$

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

End bolt resistance (continued)

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

$= \dfrac{\left(\dfrac{2.5 \cdot 1 \cdot 410.0 \cdot 24.0 \cdot 20.5}{1.25}\right)}{1000.0}$

$= 403.4\ kN$

Bearing resistance on the supporting member, $F_{b,Rd,2} = F_{b,Rd,i}$

$= 0.0\ kN$

Bearing resistance of the bolts outside the beam, $F_{b,Rd,in} = F_{b,Rd,2} \cdot n_{1,out} \cdot n_2$

$= 0.0 \cdot 1 \cdot 2$

$= 0.0\ kN$

Bearing resistance, $F_{b,Rd} = F_{b,Rd,in} + F_{b,Rd,in}$

$= 2187.9 + 0.0$

$= 2187.9\ kN$

$Shear\ capacity = F_{b,Rd}$

$= 2187.9\ kN$

Applied member shear, $V_{Ed} = 20.0\ kN$

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

$= \dfrac{20.0}{2187.9}$

$= 0.00914119$

$2187.9\ kN \geq 20.0\ kN$ **(OK)**

$0.009 \leq 1$ **(OK)**

Supported beam-flange welds (51). Reference Pg 38

Flange thickness, $t_f = 15.6\ mm$

Minimum specified total throat thickness, $a_{min} = t_f$

$= 15.6\ mm$

Minimum specified weld leg size, $s_{min} = \dfrac{\left(\dfrac{a_{min}}{0.707}\right)}{2}$

$= \dfrac{\left(\dfrac{15.6}{0.707}\right)}{2}$

$= 11.0\ mm$

$12.0\ mm \geq (s_{min} = 11.0\ mm)$ **(OK)**

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

Moment resistance of end plate connection (362). Reference Chapter 2

Number of column web doubler, $n_{dblr} = 0$
Stiffener weld size, $s_{st} = 0.0\ mm$
Column stiffener thickness, $t_{stiff} = 0.0\ mm$
Distance from top beam to the nearest inside bolt, $l_{in} = 60.0\ mm$
Distance from top beam to the nearest outside bolt, $l_{out} = 40.0\ mm$
Bolt-row edge, $e_x = 50.0\ mm$
Bolt-row spacing for the inner bolt group, $p_{inn} = 90.0\ mm$
Bolt to bolt gage, $w = 100.0\ mm$
Bolt row inside the beam, $n_{beam} = 2$
Bolt row inside the haunch, $n_{haunch} = 0$
Bolt row outside the beam flange, $n_{out} = 1$
Beam web weld size, $s_w = 8.0\ mm$
Beam flange weld size, $s_f = 12.0\ mm$
End plate yield stress, $f_{y,p} = 265.0\ MPa$
End plate thickness, $t_p = 25.0\ mm$
End plate width, $b_p = 250.0\ mm$
Haunch depth, $h_h = 0.0\ mm$
Moment on the opposite beam, $M_{b2,Ed} = -400.0\ kN \cdot m$
Beam yield stress, $f_{y,b} = 275.0\ MPa$
Beam web thickness, $t_{w,b} = 10.1\ mm$
Beam flange thickness, $t_{f,b} = 15.6\ mm$
Beam flange width, $b_{f,b} = 209.3\ mm$
Beam depth, $h_b = 533.1\ mm$
Column axial compression force, $N_{Ed} = 0.0\ kN$
Column yield stress, $f_{y,c} = 265.0\ MPa$
Distance from column end to beam tension flange, $d_{btf} = 2500.0\ mm$
Column root radius, $r_c = 12.7\ mm$
Column web thickness, $t_{w,c} = 12.8\ mm$
Column flange thickness, $t_{f,c} = 20.5\ mm$
Column flange width, $b_{f,c} = 258.8\ mm$
Column depth, $h_c = 266.7\ mm$
Column area, $A = 13638.2\ mm^2$
Applied member moment, $M_{Ed} = 400.0\ kN \cdot m$
Tension force, horizontal component, $N_{h,t,Ed} = 0.0\ kN$
Compression force, horizontal component, $N_{h,c,Ed} = 0.0\ kN$
Overall beam depth, $h_{b,overall} = h_b + h_h$
$= 533.1 + 0.0$
$= 533.1\ mm$

Moment resistance of end plate connection (362). Reference Chapter 2 (continued)

Width of the beam tension flange, $b_{f,bt} = b_{f,b}$

$= 209.3\ mm$

Thickness of the beam tension flange, $t_{f,bt} = t_{f,b}$

$= 15.6\ mm$

Thickness of the beam tension web, $t_{w,bt} = t_{w,b}$

$= 10.1\ mm$

Beam tension flange weld size, $s_{f,t} = s_f$

$= 12.0\ mm$

Beam tension web weld size, $s_{w,t} = s_w$

$= 8.0\ mm$

Width of the beam compression flange, $b_{f,bc} = b_{f,b}$

$= 209.3\ mm$

Thickness of the beam compression flange, $t_{f,bc} = t_{f,b}$

$= 15.6\ mm$

Thickness of the beam compression web, $t_{w,bc} = t_{w,b}$

$= 10.1\ mm$

Beam compression flange weld size, $s_{f,c} = s_f$

$= 12.0\ mm$

Total bolt row, $n_T = n_{out} + n_{haunch} + n_{beam}$

$= 1 + 0 + 2$

$= 3$

Bolt row below the flange, $n_{below} = n_T - n_{out}$

$= 3 - 1$

$= 2$

Bolt row above column stiffener, $n_{c,above} = 0$

Bolt row below column stiffener, $n_{c,below} = n_T$

$= 3$

$m_c = 0.5 \cdot w - 0.5 \cdot t_{w,c} - 0.8 \cdot r_c$

$= 0.5 \cdot 100.0 - 0.5 \cdot 12.8 - 0.8 \cdot 12.7$

$= 33.4\ mm$

$e_c = \frac{\left(b_{f,c} - w\right)}{2}$

$= \frac{(258.8 - 100.0)}{2}$

$= 79.4\ mm$

$e1_1 = d_{btf} - |l_{out}|$

$= 2500.0 - |40.0|$

$= 2460.0\ mm$

Moment resistance of end plate connection (362). Reference Chapter 2 (continued)

$m2_1 = \left(|l_{out}| - h_h + \frac{t_{f,bt}}{2}\right) - \frac{t_{stiff}}{2} - 0.8 \cdot s_{st}$

$= \left(|40.0| - 0.0 + \frac{15.6}{2}\right) - \frac{0.0}{2} - 0.8 \cdot 0.0$

$= 47.8\ mm$

$e1_2 = d_{btf} + |l_{in}|$

$= 2500.0 + |60.0|$

$= 2560.0\ mm$

$m2_2 = \left|h_h - |l_{in}|\right| - \frac{t_{f,bt}}{2} - \frac{t_{stiff}}{2} - 0.8 \cdot s_{st}$

$= \left|0.0 - |60.0|\right| - \frac{15.6}{2} - \frac{0.0}{2} - 0.8 \cdot 0.0$

$= 52.2\ mm$

$m2_{1,h} = 0.0\ mm$

$m2_{2,h} = 0.0\ mm$

$m2_b = |l_{in}| - t_{f,b} - 0.8 \cdot s_f$

$= |60.0| - 15.6 - 0.8 \cdot 12.0$

$= 34.8\ mm$

$m_x = |l_{out}| - 0.8 \cdot s_{f,t}$

$= |40.0| - 0.8 \cdot 12.0$

$= 30.4\ mm$

$m_p = 0.5 \cdot w - 0.5 \cdot t_{w,bt} - 0.8 \cdot s_{w,t}$

$= 0.5 \cdot 100.0 - 0.5 \cdot 10.1 - 0.8 \cdot 8.0$

$= 38.5\ mm$

$e_p = \frac{(b_p - w)}{2}$

$= \frac{(250.0 - 100.0)}{2}$

$= 75.0\ mm$

$n_c = min\ (e_c, e_p, 1.25 \cdot m_c)$

$= min\ (79.4, 75.0, 1.25 \cdot 33.4)$

$= 41.8\ mm$

$n_p = min\ (e_c, e_p, 1.25 \cdot m_p)$

$= min\ (79.4, 75.0, 1.25 \cdot 38.5)$

$= 48.2\ mm$

Row spacing, $p = p_{inn}$

$= 90.0\ mm$

Distance from bolt-row 1 to the centre of compression, $h_1 = 565.3\ mm$

Distance from bolt-row 2 to the centre of compression, $h_2 = 465.3 - p_{inn} \cdot (1 - n_{out})$

$= 465.3 - 90.0 \cdot (1 - 1)$

$= 465.3\ mm$

Moment resistance of end plate connection (362). Reference Chapter 2 (continued)

Distance from bolt-row 3 to the centre of compression, $h_3 = 465.3 - p_{inn} \cdot (2 - n_{out})$
$= 465.3 - 90.0 \cdot (2 - 1)$
$= 375.3\ mm$

Bolt-row spacing, $p_1 = h_1 - h_2$
$= 565.3 - 465.3$
$= 100.0\ mm$

Bolt-row spacing, $p_2 = h_2 - h_3$
$= 465.3 - 375.3$
$= 90.0\ mm$

Effective lengths for column flange

$n = n_{c,above} + n_{c,below}$
$= 0 + 3$
$= 3$

$n = n$
$= 3$

$m = m_c$
$= 33.4\ mm$

$e = e_c$
$= 79.4\ mm$

$m2_1 = m2_2$
$= 52.2\ mm$

$m2_2 = 0.0\ mm$

$e_1 = e1_1$
$= 2460.0\ mm$

Bolt-row spacing, $p_1 = p_1$
$= 100.0\ mm$

Bolt-row spacing, $p_2 = p_2$
$= 90.0\ mm$

Bolt-row spacing, $p_3 = p_3$
$= 0.0\ mm$

$l = 2 \cdot 3.14159 \cdot m$
$= 2 \cdot 3.14159 \cdot 33.4$
$= 210.1\ mm$

$l_{eff,cp,l}(1,1) = l$
$= 210.1\ mm$

$l = 4 \cdot m + 1.25 \cdot e$
$= 4 \cdot 33.4 + 1.25 \cdot 79.4$
$= 233.0\ mm$

Effective lengths for column flange (continued)

$l_{eff,nc,l}(1,1) = l$
$= 233.0\ mm$
$l = 2 \cdot 3.14159 \cdot m$
$= 2 \cdot 3.14159 \cdot 33.4$
$= 210.1\ mm$
$l_{eff,cp,l}(2,1) = l$
$= 210.1\ mm$
$l = 4 \cdot m + 1.25 \cdot e$
$= 4 \cdot 33.4 + 1.25 \cdot 79.4$
$= 233.0\ mm$
$l_{eff,nc,l}(2,1) = l$
$= 233.0\ mm$
$l = 2 \cdot 3.14159 \cdot m$
$= 2 \cdot 3.14159 \cdot 33.4$
$= 210.1\ mm$
$l_{eff,cp,l}(3,1) = l$
$= 210.1\ mm$
$l = 4 \cdot m + 1.25 \cdot e$
$= 4 \cdot 33.4 + 1.25 \cdot 79.4$
$= 233.0\ mm$
$l_{eff,nc,l}(3,1) = l$
$= 233.0\ mm$
$l = 3.14159 \cdot m + p_1$
$= 3.14159 \cdot 33.4 + 100.0$
$= 205.1\ mm$
$l_{eff,cp,l}(2,2) = l_{eff,cp,l}(2,2) + l$
$= 0.0 + 205.1$
$= 205.1\ mm$
$l = 2 \cdot m + 0.625 \cdot e + 0.5 \cdot p_1$
$= 2 \cdot 33.4 + 0.625 \cdot 79.4 + 0.5 \cdot 100.0$
$= 166.5\ mm$
$l_{eff,nc,l}(2,2) = l_{eff,nc,l}(2,2) + l$
$= 0.0 + 166.5$
$= 166.5\ mm$
$l = 3.14159 \cdot m + p_1$
$= 3.14159 \cdot 33.4 + 100.0$
$= 205.1\ mm$
$l_{eff,cp,l}(2,2) = l_{eff,cp,l}(2,2) + l$

Effective lengths for column flange (continued)

$= 205.1 + 205.1$

$= 410.1\ mm$

$l = 2 \cdot m + 0.625 \cdot e + 0.5 \cdot p_1$

$= 2 \cdot 33.4 + 0.625 \cdot 79.4 + 0.5 \cdot 100.0$

$= 166.5\ mm$

$l_{eff,nc,l}(2,2) = l_{eff,nc,l}(2,2) + l$

$= 166.5 + 166.5$

$= 333.0\ mm$

$l = 3.14159 \cdot m + p_2$

$= 3.14159 \cdot 33.4 + 90.0$

$= 195.1\ mm$

$l_{eff,cp,l}(3,2) = l_{eff,cp,l}(3,2) + l$

$= 0.0 + 195.1$

$= 195.1\ mm$

$l = 2 \cdot m + 0.625 \cdot e + 0.5 \cdot p_2$

$= 2 \cdot 33.4 + 0.625 \cdot 79.4 + 0.5 \cdot 90.0$

$= 161.5\ mm$

$l_{eff,nc,l}(3,2) = l_{eff,nc,l}(3,2) + l$

$= 0.0 + 161.5$

$= 161.5\ mm$

$l = 3.14159 \cdot m + p_2$

$= 3.14159 \cdot 33.4 + 90.0$

$= 195.1\ mm$

$l_{eff,cp,l}(3,2) = l_{eff,cp,l}(3,2) + l$

$= 195.1 + 195.1$

$= 390.1\ mm$

$l = 2 \cdot m + 0.625 \cdot e + 0.5 \cdot p_2$

$= 2 \cdot 33.4 + 0.625 \cdot 79.4 + 0.5 \cdot 90.0$

$= 161.5\ mm$

$l_{eff,nc,l}(3,2) = l_{eff,nc,l}(3,2) + l$

$= 161.5 + 161.5$

$= 323.0\ mm$

$l = 3.14159 \cdot m + p_1$

$= 3.14159 \cdot 33.4 + 100.0$

$= 205.1\ mm$

$l_{eff,cp,l}(3,3) = l_{eff,cp,l}(3,3) + l$

$= 0.0 + 205.1$

$= 205.1\ mm$

Effective lengths for column flange (continued)

$l = 2 \cdot m + 0.625 \cdot e + 0.5 \cdot p_1$
$= 2 \cdot 33.4 + 0.625 \cdot 79.4 + 0.5 \cdot 100.0$
$= 166.5\ mm$

$l_{eff,nc,l}(3,3) = l_{eff,nc,l}(3,3) + l$
$= 0.0 + 166.5$
$= 166.5\ mm$

$l = p_1 + p_2$
$= 100.0 + 90.0$
$= 190.0\ mm$

$l_{eff,cp,l}(3,3) = l_{eff,cp,l}(3,3) + l$
$= 205.1 + 190.0$
$= 395.1\ mm$

$l = 0.5 \cdot (p_1 + p_2)$
$= 0.5 \cdot (100.0 + 90.0)$
$= 95.0\ mm$

$l_{eff,nc,l}(3,3) = l_{eff,nc,l}(3,3) + l$
$= 166.5 + 95.0$
$= 261.5\ mm$

$l = 3.14159 \cdot m + p_2$
$= 3.14159 \cdot 33.4 + 90.0$
$= 195.1\ mm$

$l_{eff,cp,l}(3,3) = l_{eff,cp,l}(3,3) + l$
$= 395.1 + 195.1$
$= 590.1\ mm$

$l = 2 \cdot m + 0.625 \cdot e + 0.5 \cdot p_2$
$= 2 \cdot 33.4 + 0.625 \cdot 79.4 + 0.5 \cdot 90.0$
$= 161.5\ mm$

$l_{eff,nc,l}(3,3) = l_{eff,nc,l}(3,3) + l$
$= 261.5 + 161.5$
$= 423.0\ mm$

$l_{eff,1,l}(1,1) = min\ (l_{eff,cp,l}(1,1), l_{eff,nc,l}(1,1))$
$= min\ (210.1, 233.0)$
$= 210.1\ mm$

$l_{eff,2,l}(1,1) = l_{eff,nc,l}(1,1)$
$= 233.0\ mm$

$l_{eff,1,l}(2,1) = min\ (l_{eff,cp,l}(2,1), l_{eff,nc,l}(2,1))$
$= min\ (210.1, 233.0)$
$= 210.1\ mm$

Effective lengths for column flange (continued)

$l_{eff,2,l}(2,1) = l_{eff,nc,l}(2,1)$
$= 233.0\ mm$

$l_{eff,1,l}(2,2) = min\ (l_{eff,cp,l}(2,2), l_{eff,nc,l}(2,2))$
$= min\ (410.1, 333.0)$
$= 333.0\ mm$

$l_{eff,2,l}(2,2) = l_{eff,nc,l}(2,2)$
$= 333.0\ mm$

$l_{eff,1,l}(3,1) = min\ (l_{eff,cp,l}(3,1), l_{eff,nc,l}(3,1))$
$= min\ (210.1, 233.0)$
$= 210.1\ mm$

$l_{eff,2,l}(3,1) = l_{eff,nc,l}(3,1)$
$= 233.0\ mm$

$l_{eff,1,l}(3,2) = min\ (l_{eff,cp,l}(3,2), l_{eff,nc,l}(3,2))$
$= min\ (390.1, 323.0)$
$= 323.0\ mm$

$l_{eff,2,l}(3,2) = l_{eff,nc,l}(3,2)$
$= 323.0\ mm$

$l_{eff,1,l}(3,3) = min\ (l_{eff,cp,l}(3,3), l_{eff,nc,l}(3,3))$
$= min\ (590.1, 423.0)$
$= 423.0\ mm$

$l_{eff,2,l}(3,3) = l_{eff,nc,l}(3,3)$
$= 423.0\ mm$

$l_{eff,1}(1,1) = l_{eff,1,l}(1,1)$
$= 210.1\ mm$

$l_{eff,2}(1,1) = l_{eff,2,l}(1,1)$
$= 233.0\ mm$

$l_{eff,1}(2,1) = l_{eff,1,l}(2,1)$
$= 210.1\ mm$

$l_{eff,2}(2,1) = l_{eff,2,l}(2,1)$
$= 233.0\ mm$

$l_{eff,1}(2,2) = l_{eff,1,l}(2,2)$
$= 333.0\ mm$

$l_{eff,2}(2,2) = l_{eff,2,l}(2,2)$
$= 333.0\ mm$

$l_{eff,1}(3,1) = l_{eff,1,l}(3,1)$
$= 210.1\ mm$

$l_{eff,2}(3,1) = l_{eff,2,l}(3,1)$
$= 233.0\ mm$

Effective lengths for column flange (continued)

$l_{eff,1}(3,2) = l_{eff,1,l}(3,2)$
$= 323.0\ mm$

$l_{eff,2}(3,2) = l_{eff,2,l}(3,2)$
$= 323.0\ mm$

$l_{eff,1}(3,3) = l_{eff,1,l}(3,3)$
$= 423.0\ mm$

$l_{eff,2}(3,3) = l_{eff,2,l}(3,3)$
$= 423.0\ mm$

Effective lengths for end plate

$n = n_{out}$
$= 1$

$m = m_p$
$= 38.5\ mm$

$e = e_p$
$= 75.0\ mm$

$m2_1 = 0.0\ mm$

$m2_2 = 0.0\ mm$

$e_1 = 0.0\ mm$

$l_{eff,cp,l}(1,1) = min\ (2 \cdot 3.14159 \cdot m_x, 3.14159 \cdot m_x + w, 3.14159 \cdot m_x + 2 \cdot e)$
$= min\ (2 \cdot 3.14159 \cdot 30.4, 3.14159 \cdot 30.4 + 100.0, 3.14159 \cdot 30.4 + 2 \cdot 75.0)$
$= 191.0\ mm$

$l_{eff,nc,l}(1,1) = min\ (4 \cdot m_x + 1.25 \cdot e_x, e + 2 \cdot m_x + 0.625 \cdot e_x, 0.5 \cdot (2 \cdot e + w) \cdot w + 2 \cdot m_x + 0.625 \cdot e_x)$
$= min\ (4 \cdot 30.4 + 1.25 \cdot 50.0, 75.0 + 2 \cdot 30.4 + 0.625 \cdot 50.0, 0.5 \cdot (2 \cdot 75.0 + 100.0) \cdot 100.0 + 2 \cdot 30.4 + 0.625 \cdot$
$= 125.0\ mm$

$l_{eff,1,l}(1,1) = min\ (l_{eff,cp,l}(1,1), l_{eff,nc,l}(1,1))$
$= min\ (191.0, 125.0)$
$= 125.0\ mm$

$l_{eff,2,l}(1,1) = l_{eff,nc,l}(1,1)$
$= 125.0\ mm$

$n = n_{beam}$
$= 2$

$m = m_p$
$= 38.5\ mm$

$e = e_p$
$= 75.0\ mm$

$m2_1 = m2_b$
$= 34.8\ mm$

$m2_2 = 0.0\ mm$

Effective lengths for end plate (continued)

$e_1 = 253974.6\ mm$

Bolt-row spacing, $p_1 = p_{inn}$

$= 90.0\ mm$

$l = 2 \cdot 3.14159 \cdot m$

$= 2 \cdot 3.14159 \cdot 38.5$

$= 242.2\ mm$

$l_{eff,cp,l}(1,1) = l$

$= 242.2\ mm$

$\alpha = 7.34745$

$l = \alpha \cdot m$

$= 7.34745 \cdot 38.5$

$= 283.2\ mm$

$l_{eff,nc,l}(1,1) = l$

$= 283.2\ mm$

$l = 2 \cdot 3.14159 \cdot m$

$= 2 \cdot 3.14159 \cdot 38.5$

$= 242.2\ mm$

$l_{eff,cp,l}(2,1) = l$

$= 242.2\ mm$

$l = 4 \cdot m + 1.25 \cdot e$

$= 4 \cdot 38.5 + 1.25 \cdot 75.0$

$= 248.0\ mm$

$l_{eff,nc,l}(2,1) = l$

$= 248.0\ mm$

$l = 3.14159 \cdot m + p_1$

$= 3.14159 \cdot 38.5 + 90.0$

$= 211.1\ mm$

$l_{eff,cp,l}(2,2) = l_{eff,cp,l}(2,2) + l$

$= 0.0 + 211.1$

$= 211.1\ mm$

$\alpha = 7.34745$

$l = (0.5 \cdot p_1 + \alpha \cdot m) - (2 \cdot m + 0.625 \cdot e)$

$= (0.5 \cdot 90.0 + 7.34745 \cdot 38.5) - (2 \cdot 38.5 + 0.625 \cdot 75.0)$

$= 204.3\ mm$

$l_{eff,nc,l}(2,2) = l_{eff,nc,l}(2,2) + l$

$= 0.0 + 204.3$

$= 204.3\ mm$

$l = 3.14159 \cdot m + p_1$

Effective lengths for end plate (continued)

$= 3.14159 \cdot 38.5 + 90.0$

$= 211.1\ mm$

$l_{eff,cp,l}(2,2) = l_{eff,cp,l}(2,2) + l$

$= 211.1 + 211.1$

$= 422.2\ mm$

$l = 2 \cdot m + 0.625 \cdot e + 0.5 \cdot p_1$

$= 2 \cdot 38.5 + 0.625 \cdot 75.0 + 0.5 \cdot 90.0$

$= 169.0\ mm$

$l_{eff,nc,l}(2,2) = l_{eff,nc,l}(2,2) + l$

$= 204.3 + 169.0$

$= 373.2\ mm$

$l_{eff,1,l}(1,1) = min\ \left(l_{eff,cp,l}(1,1), l_{eff,nc,l}(1,1)\right)$

$= min\ (242.2, 283.2)$

$= 242.2\ mm$

$l_{eff,2,l}(1,1) = l_{eff,nc,l}(1,1)$

$= 283.2\ mm$

$l_{eff,1,l}(2,1) = min\ \left(l_{eff,cp,l}(2,1), l_{eff,nc,l}(2,1)\right)$

$= min\ (242.2, 248.0)$

$= 242.2\ mm$

$l_{eff,2,l}(2,1) = l_{eff,nc,l}(2,1)$

$= 248.0\ mm$

$l_{eff,1,l}(2,2) = min\ \left(l_{eff,cp,l}(2,2), l_{eff,nc,l}(2,2)\right)$

$= min\ (422.2, 373.2)$

$= 373.2\ mm$

$l_{eff,2,l}(2,2) = l_{eff,nc,l}(2,2)$

$= 373.2\ mm$

$l_{eff,1}(1,1) = l_{eff,1,l}(1,1)$

$= 125.0\ mm$

$l_{eff,2}(1,1) = l_{eff,2,l}(1,1)$

$= 125.0\ mm$

$l_{eff,1}(2,1) = l_{eff,1,l}(1,1)$

$= 242.2\ mm$

$l_{eff,2}(2,1) = l_{eff,2,l}(1,1)$

$= 283.2\ mm$

$l_{eff,1}(3,1) = l_{eff,1,l}(2,1)$

$= 242.2\ mm$

$l_{eff,2}(3,1) = l_{eff,2,l}(2,1)$

$= 248.0\ mm$

Effective lengths for end plate (continued)

$l_{eff,1}(3,2) = l_{eff,1,l}(2,2)$

$= 373.2\ mm$

$l_{eff,2}(3,2) = l_{eff,2,l}(2,2)$

$= 373.2\ mm$

Effective design tension resistances for all the individual bolt-rows and bolt-row groups

$m = m_x$

$= 30.4\ mm$

$n = min\ (e_x, 1.25 \cdot m)$

$= min\ (50.0, 1.25 \cdot 30.4)$

$= 38.0\ mm$

Effective design tension resistances for individual bolt-row 1

Column flange in bending

Width across points of the bolt head or nut, $d_w = 40.0\ mm$

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

$= \frac{40.0}{4}$

$= 10.0\ mm$

Bolt tension area, $A_{b,t} = 352.5\ mm^2$

$k_2 = 0.9$

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

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

Design tension resistance of a bolt, $F_{t,Rd} = \frac{\left(\frac{k_2 \cdot f_{ub} \cdot A_{b,t}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$= \frac{\left(\frac{0.9 \cdot 800.0 \cdot 352.5}{1.25}\right)}{1000.0}$

$= 203.0\ kN$

$\gamma_{M0} = 1$

$M_{pl,1,Rd} = \frac{\left(\frac{0.25 \cdot l_{eff,1}(1,1) \cdot t_{f,c}^2 \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000000.0}$

$= \frac{\left(\frac{0.25 \cdot 210.1 \cdot 20.5^2 \cdot 265.0}{1}\right)}{1000000.0}$

Column flange in bending (continued)

$= 5.8\ kN \cdot m$

$$M_{pl,2,Rd} = \frac{\left(\frac{0.25 \cdot l_{eff,2}(1,1) \cdot t_{f,c}^2 \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000000.0}$$

$$= \frac{\left(\frac{0.25 \cdot 233.0 \cdot 20.5^2 \cdot 265.0}{1}\right)}{1000000.0}$$

$= 6.5\ kN \cdot m$

$$F_{T,1,Rd} = \left(\frac{(8 \cdot n_c - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{(2 \cdot m_c \cdot n_c - e_w \cdot (m_c + n_c))}\right) \cdot 1000.0$$

$$= \left(\frac{(8 \cdot 41.8 - 2 \cdot 10.0) \cdot 5.8}{(2 \cdot 33.4 \cdot 41.8 - 10.0 \cdot (33.4 + 41.8))}\right) \cdot 1000.0$$

$= 900.1\ kN$

$$F_{T,2,Rd} = \left(\frac{\left(2 \cdot M_{pl,2,Rd} + \frac{n_c \cdot 2 \cdot 1 \cdot F_{t,Rd}}{1000.0}\right)}{(m_c + n_c)}\right) \cdot 1000.0$$

$$= \left(\frac{\left(2 \cdot 6.5 + \frac{41.8 \cdot 2 \cdot 1 \cdot 203.0}{1000.0}\right)}{(33.4 + 41.8)}\right) \cdot 1000.0$$

$= 398.0\ kN$

$F_{T,3,Rd} = 2 \cdot 1 \cdot F_{t,Rd}$

$= 2 \cdot 1 \cdot 203.0$

$= 406.1\ kN$

Design resistance of column flange in bending, $F_{t,fc,Rd} = min\ (F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$= min\ (900.1, 398.0, 406.1)$

$= 398.0\ kN$

Column web in tension

Effective width of column web in tension, $l_{eff,t,wc} = l_{eff,2}(1,1)$

$= 233.0\ mm$

Effective thickness of column web, $t_{wc,eff} = t_{w,c} + 0.5 \cdot t_{w,c} \cdot n_{dblr}$

$= 12.8 + 0.5 \cdot 12.8 \cdot 0$

$= 12.8\ mm$

Calculation of reduction factor for shear interaction

Transformation parameter, $\beta = min\left(\left|1 + \frac{M_{b2,Ed}}{M_{Ed}}\right|\right)$

$= min\left(\left|1 + \frac{-400.0}{400.0}\right|\right)$

$= 0$

Shear area of the column, $A_{vc} = A - 2 \cdot b_{f,c} \cdot t_{f,c} + (t_{w,c} + 2 \cdot r_c) \cdot t_{f,c}$

$= 13638.2 - 2 \cdot 258.8 \cdot 20.5 + (12.8 + 2 \cdot 12.7) \cdot 20.5$

$= 3810.5\ mm^2$

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{l_{eff,t,wc} \cdot t_{wc,eff}}{A_{vc}}\right)^2}}$$

$$= \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{233.0 \cdot 12.8}{3810.5}\right)^2}}$$

$= 0.746097$

$$\omega_2 = \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{l_{eff,t,wc} \cdot t_{wc,eff}}{A_{vc}}\right)^2}}$$

$$= \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{233.0 \cdot 12.8}{3810.5}\right)^2}}$$

$= 0.488783$

Reduction factor for shear interaction, $\omega = 1$

$\gamma_{M0} = 1$

Design resistance of column web in tension, $F_{t,wc,Rd} = \frac{\left(\frac{\omega \cdot l_{eff,t,wc} \cdot t_{wc,eff} \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000.0}$

$$= \frac{\left(\frac{1 \cdot 233.0 \cdot 12.8 \cdot 265.0}{1}\right)}{1000.0}$$

$= 790.4\ kN$

End plate in bending

Width across points of the bolt head or nut, $d_w = 40.0\ mm$

$$e_w = \frac{d_w}{4} = \frac{40.0}{4} = 10.0\ mm$$

Bolt tension area, $A_{b,t} = 352.5\ mm^2$

$k_2 = 0.9$

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

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

Design tension resistance of a bolt, $F_{t,Rd} = \dfrac{\left(\dfrac{k_2 \cdot f_{ub} \cdot A_{b,t}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$$= \frac{\left(\dfrac{0.9 \cdot 800.0 \cdot 352.5}{1.25}\right)}{1000.0}$$

$$= 203.0\ kN$$

$\gamma_{M0} = 1$

$$M_{pl,1,Rd} = \frac{\left(\dfrac{0.25 \cdot l_{eff,1}(1,1) \cdot t_p^2 \cdot f_{y,p}}{\gamma_{M0}}\right)}{1000000.0}$$

$$= \frac{\left(\dfrac{0.25 \cdot 125.0 \cdot 25.0^2 \cdot 265.0}{1}\right)}{1000000.0}$$

$$= 5.2\ kN \cdot m$$

$$M_{pl,2,Rd} = M_{pl,1,Rd} = 5.2\ kN \cdot m$$

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

$$= \left(\frac{(8 \cdot 38.0 - 2 \cdot 10.0) \cdot 5.2}{(2 \cdot 30.4 \cdot 38.0 - 10.0 \cdot (30.4 + 38.0))}\right) \cdot 1000.0$$

$$= 903.8\ kN$$

$$F_{T,2,Rd} = \left(\frac{\left(2 \cdot M_{pl,2,Rd} + \dfrac{n \cdot 2 \cdot 1 \cdot F_{t,Rd}}{1000.0}\right)}{(m + n)}\right) \cdot 1000.0$$

End plate in bending (continued)

$$= \left(\frac{\left(2 \cdot 5.2 + \frac{38.0 \cdot 2 \cdot 1 \cdot 203.0}{1000.0}\right)}{(30.4 + 38.0)} \right) \cdot 1000.0$$

$= 376.9\ kN$

$F_{T,3,Rd} = 2 \cdot 1 \cdot F_{t,Rd}$

$= 2 \cdot 1 \cdot 203.0$

$= 406.1\ kN$

Design resistance of end plate in bending, $F_{t,ep,Rd} = min\ (F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$= min\ (903.8, 376.9, 406.1)$

$= 376.9\ kN$

$F_{t,Rd,EL}(1,1) = F_{t,fc,Rd}$

$= 398.0\ kN$

$F_{t,Rd,EL}(1,1) = F_{t,ep,Rd}$

$= 376.9\ kN$

$m = m_p$

$= 38.5\ mm$

$n = n_p$

$= 48.2\ mm$

Effective design tension resistances for individual bolt-row 2

Column flange in bending

Width across points of the bolt head or nut, $d_w = 40.0\ mm$

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

$= \frac{40.0}{4}$

$= 10.0\ mm$

Bolt tension area, $A_{b,t} = 352.5\ mm^2$

$k_2 = 0.9$

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

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

Design tension resistance of a bolt, $F_{t,Rd} = \frac{\left(\frac{k_2 \cdot f_{ub} \cdot A_{b,t}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$$= \frac{\left(\frac{0.9 \cdot 800.0 \cdot 352.5}{1.25}\right)}{1000.0}$$

$= 203.0\ kN$

$\gamma_{M0} = 1$

Column flange in bending (continued)

$$M_{pl,1,Rd} = \frac{\left(\frac{0.25 \cdot l_{eff,1}(2,1) \cdot t_{f,c}^2 \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000000.0}$$

$$= \frac{\left(\frac{0.25 \cdot 210.1 \cdot 20.5^2 \cdot 265.0}{1}\right)}{1000000.0}$$

$$= 5.8\ kN \cdot m$$

$$M_{pl,2,Rd} = \frac{\left(\frac{0.25 \cdot l_{eff,2}(2,1) \cdot t_{f,c}^2 \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000000.0}$$

$$= \frac{\left(\frac{0.25 \cdot 233.0 \cdot 20.5^2 \cdot 265.0}{1}\right)}{1000000.0}$$

$$= 6.5\ kN \cdot m$$

$$F_{T,1,Rd} = \left(\frac{(8 \cdot n_c - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{(2 \cdot m_c \cdot n_c - e_w \cdot (m_c + n_c))}\right) \cdot 1000.0$$

$$= \left(\frac{(8 \cdot 41.8 - 2 \cdot 10.0) \cdot 5.8}{(2 \cdot 33.4 \cdot 41.8 - 10.0 \cdot (33.4 + 41.8))}\right) \cdot 1000.0$$

$$= 900.1\ kN$$

$$F_{T,2,Rd} = \left(\frac{\left(2 \cdot M_{pl,2,Rd} + \frac{n_c \cdot 2 \cdot 1 \cdot F_{t,Rd}}{1000.0}\right)}{(m_c + n_c)}\right) \cdot 1000.0$$

$$= \left(\frac{\left(2 \cdot 6.5 + \frac{41.8 \cdot 2 \cdot 1 \cdot 203.0}{1000.0}\right)}{(33.4 + 41.8)}\right) \cdot 1000.0$$

$$= 398.0\ kN$$

$$F_{T,3,Rd} = 2 \cdot 1 \cdot F_{t,Rd}$$

$$= 2 \cdot 1 \cdot 203.0$$

$$= 406.1\ kN$$

Design resistance of column flange in bending, $F_{t,fc,Rd} = min\ (F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$$= min\ (900.1, 398.0, 406.1)$$

$$= 398.0\ kN$$

Column web in tension

Effective width of column web in tension, $l_{eff,t,wc} = l_{eff,2}(2,1)$

$= 233.0\ mm$

Effective thickness of column web, $t_{wc,eff} = t_{w,c} + 0.5 \cdot t_{w,c} \cdot n_{dblr}$

$= 12.8 + 0.5 \cdot 12.8 \cdot 0$

$= 12.8\ mm$

Calculation of reduction factor for shear interaction

Transformation parameter, $\beta = min\left(\left|1 + \frac{M_{b2,Ed}}{M_{Ed}}\right|\right)$

$= min\left(\left|1 + \frac{-400.0}{400.0}\right|\right)$

$= 0$

Shear area of the column, $A_{vc} = A - 2 \cdot b_{f,c} \cdot t_{f,c} + (t_{w,c} + 2 \cdot r_c) \cdot t_{f,c}$

$= 13638.2 - 2 \cdot 258.8 \cdot 20.5 + (12.8 + 2 \cdot 12.7) \cdot 20.5$

$= 3810.5\ mm^2$

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{l_{eff,t,wc} \cdot t_{wc,eff}}{A_{vc}}\right)^2}}$$

$$= \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{233.0 \cdot 12.8}{3810.5}\right)^2}}$$

$= 0.746097$

$$\omega_2 = \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{l_{eff,t,wc} \cdot t_{wc,eff}}{A_{vc}}\right)^2}}$$

$$= \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{233.0 \cdot 12.8}{3810.5}\right)^2}}$$

$= 0.488783$

Reduction factor for shear interaction, $\omega = 1$

$\gamma_{M0} = 1$

Column web in tension (continued)

Design resistance of column web in tension, $F_{t,wc,Rd} = \frac{\left(\frac{\omega \cdot l_{eff,t,wc} \cdot t_{wc,eff} \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000.0}$

$= \frac{\left(\frac{1 \cdot 233.0 \cdot 12.8 \cdot 265.0}{1}\right)}{1000.0}$

$= 790.4\ kN$

End plate in bending

Width across points of the bolt head or nut, $d_w = 40.0\ mm$

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

$= \frac{40.0}{4}$

$= 10.0\ mm$

Bolt tension area, $A_{b,t} = 352.5\ mm^2$

$k_2 = 0.9$

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

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

Design tension resistance of a bolt, $F_{t,Rd} = \frac{\left(\frac{k_2 \cdot f_{ub} \cdot A_{b,t}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$= \frac{\left(\frac{0.9 \cdot 800.0 \cdot 352.5}{1.25}\right)}{1000.0}$

$= 203.0\ kN$

$\gamma_{M0} = 1$

$M_{pl,1,Rd} = \frac{\left(\frac{0.25 \cdot l_{eff,1}(2,1) \cdot t_p^2 \cdot f_{y,p}}{\gamma_{M0}}\right)}{1000000.0}$

$= \frac{\left(\frac{0.25 \cdot 242.2 \cdot 25.0^2 \cdot 265.0}{1}\right)}{1000000.0}$

$= 10.0\ kN \cdot m$

End plate in bending (continued)

$$M_{pl,2,Rd} = \frac{\left(\frac{0.25 \cdot l_{eff,2}(2,1) \cdot t_p^2 \cdot f_{y,p}}{\gamma_{M0}}\right)}{1000000.0}$$

$$= \frac{\left(\frac{0.25 \cdot 283.2 \cdot 25.0^2 \cdot 265.0}{1}\right)}{1000000.0}$$

$$= 11.7\ kN \cdot m$$

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

$$= \left(\frac{(8 \cdot 48.2 - 2 \cdot 10.0) \cdot 10.0}{(2 \cdot 38.5 \cdot 48.2 - 10.0 \cdot (38.5 + 48.2))}\right) \cdot 1000.0$$

$$= 1287.2\ kN$$

$$F_{T,2,Rd} = \left(\frac{\left(2 \cdot M_{pl,2,Rd} + \frac{n \cdot 2 \cdot 1 \cdot F_{t,Rd}}{1000.0}\right)}{(m + n)}\right) \cdot 1000.0$$

$$= \left(\frac{\left(2 \cdot 11.7 + \frac{48.2 \cdot 2 \cdot 1 \cdot 203.0}{1000.0}\right)}{(38.5 + 48.2)}\right) \cdot 1000.0$$

$$= 496.0\ kN$$

$$F_{T,3,Rd} = 2 \cdot 1 \cdot F_{t,Rd}$$

$$= 2 \cdot 1 \cdot 203.0$$

$$= 406.1\ kN$$

Design resistance of end plate in bending, $F_{t,ep,Rd} = min\ (F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$$= min\ (1287.2, 496.0, 406.1)$$

$$= 406.1\ kN$$

Beam web in tension

Effective width of beam web in tension, $l_{eff,t,wb} = min\ (l_{eff,1}(2,1), l_{eff,2}(2,1))$

$$= min\ (242.2, 283.2)$$

$$= 242.2\ mm$$

$$\gamma_{M0} = 1$$

Design resistance of beam web in tension, $F_{t,wb,Rd} = \frac{\left(\frac{l_{eff,t,wb} \cdot t_{w,bt} \cdot f_{y,b}}{\gamma_{M0}}\right)}{1000.0}$

Beam web in tension (continued)

$$= \frac{\left(\frac{242.2 \cdot 10.1 \cdot 275.0}{1}\right)}{1000.0}$$

$$= 672.8\ kN$$

$F_{t,Rd,EL}(2,1) = F_{t,fc,Rd}$

$= 398.0\ kN$

Effective design tension resistances for bolt-row group with bolt-rows 1-2

Column flange in bending

Width across points of the bolt head or nut, $d_w = 40.0\ mm$

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

$$= \frac{40.0}{4}$$

$$= 10.0\ mm$$

Bolt tension area, $A_{b,t} = 352.5\ mm^2$

$k_2 = 0.9$

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

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

Design tension resistance of a bolt, $F_{t,Rd} = \frac{\left(\frac{k_2 \cdot f_{ub} \cdot A_{b,t}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$$= \frac{\left(\frac{0.9 \cdot 800.0 \cdot 352.5}{1.25}\right)}{1000.0}$$

$$= 203.0\ kN$$

$\gamma_{M0} = 1$

$$M_{pl,1,Rd} = \frac{\left(\frac{0.25 \cdot l_{eff,1}(2,2) \cdot t_{f,c}^2 \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000000.0}$$

$$= \frac{\left(\frac{0.25 \cdot 333.0 \cdot 20.5^2 \cdot 265.0}{1}\right)}{1000000.0}$$

$$= 9.3\ kN \cdot m$$

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

$= 9.3\ kN \cdot m$

Column flange in bending (continued)

$$F_{T,1,Rd} = \left(\frac{(8 \cdot n_c - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{(2 \cdot m_c \cdot n_c - e_w \cdot (m_c + n_c))} \right) \cdot 1000.0$$

$$= \left(\frac{(8 \cdot 41.8 - 2 \cdot 10.0) \cdot 9.3}{(2 \cdot 33.4 \cdot 41.8 - 10.0 \cdot (33.4 + 41.8))} \right) \cdot 1000.0$$

$$= 1426.7\ kN$$

$$F_{T,2,Rd} = \left(\frac{\left(2 \cdot M_{pl,2,Rd} + \frac{n_c \cdot 2 \cdot 2 \cdot F_{t,Rd}}{1000.0}\right)}{(m_c + n_c)} \right) \cdot 1000.0$$

$$= \left(\frac{\left(2 \cdot 9.3 + \frac{41.8 \cdot 2 \cdot 2 \cdot 203.0}{1000.0}\right)}{(33.4 + 41.8)} \right) \cdot 1000.0$$

$$= 697.7\ kN$$

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

$$= 2 \cdot 2 \cdot 203.0$$

$$= 812.2\ kN$$

Design resistance of column flange in bending, $F_{t,fc,Rd} = min\ (F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$$= min\ (1426.7, 697.7, 812.2)$$

$$= 697.7\ kN$$

Column web in tension

Effective width of column web in tension, $l_{eff,t,wc} = l_{eff,2}(2,2)$

$$= 333.0\ mm$$

Effective thickness of column web, $t_{wc,eff} = t_{w,c} + 0.5 \cdot t_{w,c} \cdot n_{dblr}$

$$= 12.8 + 0.5 \cdot 12.8 \cdot 0$$

$$= 12.8\ mm$$

Calculation of reduction factor for shear interaction

Transformation parameter, $\beta = min\ \left(\left| 1 + \frac{M_{b2,Ed}}{M_{Ed}} \right| \right)$

$$= min\ \left(\left| 1 + \frac{-400.0}{400.0} \right| \right)$$

$$= 0$$

Shear area of the column, $A_{vc} = A - 2 \cdot b_{f,c} \cdot t_{f,c} + (t_{w,c} + 2 \cdot r_c) \cdot t_{f,c}$

$$= 13638.2 - 2 \cdot 258.8 \cdot 20.5 + (12.8 + 2 \cdot 12.7) \cdot 20.5$$

$$= 3810.5\ mm^2$$

Calculation of reduction factor for shear interaction (continued)

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{l_{eff,t,wc} \cdot t_{wc,eff}}{A_{vc}}\right)^2}}$$

$$= \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{333.0 \cdot 12.8}{3810.5}\right)^2}}$$

$$= 0.617012$$

$$\omega_2 = \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{l_{eff,t,wc} \cdot t_{wc,eff}}{A_{vc}}\right)^2}}$$

$$= \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{333.0 \cdot 12.8}{3810.5}\right)^2}}$$

$$= 0.364982$$

Reduction factor for shear interaction, $\omega = 1$

$\gamma_{M0} = 1$

Design resistance of column web in tension, $F_{t,wc,Rd} = \frac{\left(\frac{\omega \cdot l_{eff,t,wc} \cdot t_{wc,eff} \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000.0}$

$$= \frac{\left(\frac{1 \cdot 333.0 \cdot 12.8 \cdot 265.0}{1}\right)}{1000.0}$$

$$= 1129.6\ kN$$

$F_{t,Rd,EL}(2,2) = F_{t,fc,Rd}$

$= 697.7\ kN$

$m = m_p$

$= 38.5\ mm$

$n = n_p$

$= 48.2\ mm$

Effective design tension resistances for individual bolt-row 3

Column flange in bending

Width across points of the bolt head or nut, $d_w = 40.0\ mm$

$$e_w = \frac{d_w}{4} = \frac{40.0}{4} = 10.0\ mm$$

Bolt tension area, $A_{b,t} = 352.5\ mm^2$

$k_2 = 0.9$

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

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

Design tension resistance of a bolt, $F_{t,Rd} = \frac{\left(\frac{k_2 \cdot f_{ub} \cdot A_{b,t}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$$= \frac{\left(\frac{0.9 \cdot 800.0 \cdot 352.5}{1.25}\right)}{1000.0}$$

$$= 203.0\ kN$$

$\gamma_{M0} = 1$

$$M_{pl,1,Rd} = \frac{\left(\frac{0.25 \cdot l_{eff,1}(3,1) \cdot t_{f,c}^2 \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000000.0}$$

$$= \frac{\left(\frac{0.25 \cdot 210.1 \cdot 20.5^2 \cdot 265.0}{1}\right)}{1000000.0}$$

$$= 5.8\ kN \cdot m$$

$$M_{pl,2,Rd} = \frac{\left(\frac{0.25 \cdot l_{eff,2}(3,1) \cdot t_{f,c}^2 \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000000.0}$$

$$= \frac{\left(\frac{0.25 \cdot 233.0 \cdot 20.5^2 \cdot 265.0}{1}\right)}{1000000.0}$$

$$= 6.5\ kN \cdot m$$

Column flange in bending (continued)

$$F_{T,1,Rd} = \left(\frac{(8 \cdot n_c - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{(2 \cdot m_c \cdot n_c - e_w \cdot (m_c + n_c))}\right) \cdot 1000.0$$
$$= \left(\frac{(8 \cdot 41.8 - 2 \cdot 10.0) \cdot 5.8}{(2 \cdot 33.4 \cdot 41.8 - 10.0 \cdot (33.4 + 41.8))}\right) \cdot 1000.0$$
$$= 900.1\ kN$$

$$F_{T,2,Rd} = \left(\frac{\left(2 \cdot M_{pl,2,Rd} + \frac{n_c \cdot 2 \cdot 1 \cdot F_{t,Rd}}{1000.0}\right)}{(m_c + n_c)}\right) \cdot 1000.0$$
$$= \left(\frac{\left(2 \cdot 6.5 + \frac{41.8 \cdot 2 \cdot 1 \cdot 203.0}{1000.0}\right)}{(33.4 + 41.8)}\right) \cdot 1000.0$$
$$= 398.0\ kN$$

$$F_{T,3,Rd} = 2 \cdot 1 \cdot F_{t,Rd}$$
$$= 2 \cdot 1 \cdot 203.0$$
$$= 406.1\ kN$$

Design resistance of column flange in bending, $F_{t,fc,Rd} = min\ (F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$
$$= min\ (900.1, 398.0, 406.1)$$
$$= 398.0\ kN$$

Column web in tension

Effective width of column web in tension, $l_{eff,t,wc} = l_{eff,2}(3,1)$
$$= 233.0\ mm$$

Effective thickness of column web, $t_{wc,eff} = t_{w,c} + 0.5 \cdot t_{w,c} \cdot n_{dblr}$
$$= 12.8 + 0.5 \cdot 12.8 \cdot 0$$
$$= 12.8\ mm$$

Calculation of reduction factor for shear interaction

Transformation parameter, $\beta = min\ \left(\left|1 + \frac{M_{b2,Ed}}{M_{Ed}}\right|\right)$
$$= min\ \left(\left|1 + \frac{-400.0}{400.0}\right|\right)$$
$$= 0$$

Shear area of the column, $A_{vc} = A - 2 \cdot b_{f,c} \cdot t_{f,c} + (t_{w,c} + 2 \cdot r_c) \cdot t_{f,c}$
$$= 13638.2 - 2 \cdot 258.8 \cdot 20.5 + (12.8 + 2 \cdot 12.7) \cdot 20.5$$
$$= 3810.5\ mm^2$$

Calculation of reduction factor for shear interaction (continued)

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{l_{eff,t,wc} \cdot t_{wc,eff}}{A_{vc}} \right)^2}}$$

$$= \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{233.0 \cdot 12.8}{3810.5} \right)^2}}$$

$$= 0.746097$$

$$\omega_2 = \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{l_{eff,t,wc} \cdot t_{wc,eff}}{A_{vc}} \right)^2}}$$

$$= \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{233.0 \cdot 12.8}{3810.5} \right)^2}}$$

$$= 0.488783$$

Reduction factor for shear interaction, $\omega = 1$

$\gamma_{M0} = 1$

Design resistance of column web in tension, $F_{t,wc,Rd} = \frac{\left(\frac{\omega \cdot l_{eff,t,wc} \cdot t_{wc,eff} \cdot f_{y,c}}{\gamma_{M0}} \right)}{1000.0}$

$$= \frac{\left(\frac{1 \cdot 233.0 \cdot 12.8 \cdot 265.0}{1} \right)}{1000.0}$$

$$= 790.4\ kN$$

End plate in bending

Width across points of the bolt head or nut, $d_w = 40.0\ mm$

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

$$= \frac{40.0}{4}$$

$$= 10.0\ mm$$

Bolt tension area, $A_{b,t} = 352.5\ mm^2$

$k_2 = 0.9$

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

End plate in bending (continued)

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

Design tension resistance of a bolt, $F_{t,Rd} = \dfrac{\left(\dfrac{k_2 \cdot f_{ub} \cdot A_{b,t}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{0.9 \cdot 800.0 \cdot 352.5}{1.25}\right)}{1000.0}$

$= 203.0\ kN$

$\gamma_{M0} = 1$

$M_{pl,1,Rd} = \dfrac{\left(\dfrac{0.25 \cdot l_{eff,1}(3,1) \cdot t_p^2 \cdot f_{y,p}}{\gamma_{M0}}\right)}{1000000.0}$

$= \dfrac{\left(\dfrac{0.25 \cdot 242.2 \cdot 25.0^2 \cdot 265.0}{1}\right)}{1000000.0}$

$= 10.0\ kN \cdot m$

$M_{pl,2,Rd} = \dfrac{\left(\dfrac{0.25 \cdot l_{eff,2}(3,1) \cdot t_p^2 \cdot f_{y,p}}{\gamma_{M0}}\right)}{1000000.0}$

$= \dfrac{\left(\dfrac{0.25 \cdot 248.0 \cdot 25.0^2 \cdot 265.0}{1}\right)}{1000000.0}$

$= 10.3\ kN \cdot m$

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

$= \left(\dfrac{(8 \cdot 48.2 - 2 \cdot 10.0) \cdot 10.0}{(2 \cdot 38.5 \cdot 48.2 - 10.0 \cdot (38.5 + 48.2))}\right) \cdot 1000.0$

$= 1287.2\ kN$

$F_{T,2,Rd} = \left(\dfrac{\left(2 \cdot M_{pl,2,Rd} + \dfrac{n \cdot 2 \cdot 1 \cdot F_{t,Rd}}{1000.0}\right)}{(m + n)}\right) \cdot 1000.0$

End plate in bending (continued)

$$= \left(\frac{\left(2 \cdot 10.3 + \frac{48.2 \cdot 2 \cdot 1 \cdot 203.0}{1000.0}\right)}{(38.5 + 48.2)} \right) \cdot 1000.0$$

$= 462.3\ kN$

$F_{T,3,Rd} = 2 \cdot 1 \cdot F_{t,Rd}$

$= 2 \cdot 1 \cdot 203.0$

$= 406.1\ kN$

Design resistance of end plate in bending, $F_{t,ep,Rd} = min\ (F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$= min\ (1287.2, 462.3, 406.1)$

$= 406.1\ kN$

Beam web in tension

Effective width of beam web in tension, $l_{eff,t,wb} = min\ (l_{eff,1}(3,1), l_{eff,2}(3,1))$

$= min\ (242.2, 248.0)$

$= 242.2\ mm$

$\gamma_{M0} = 1$

Design resistance of beam web in tension, $F_{t,wb,Rd} = \dfrac{\left(\dfrac{l_{eff,t,wb} \cdot t_{w,bt} \cdot f_{y,b}}{\gamma_{M0}}\right)}{1000.0}$

$$= \frac{\left(\frac{242.2 \cdot 10.1 \cdot 275.0}{1}\right)}{1000.0}$$

$= 672.8\ kN$

$F_{t,Rd,EL}(3,1) = F_{t,fc,Rd}$

$= 398.0\ kN$

Effective design tension resistances for bolt-row group with bolt-rows 2-3

Column flange in bending

Width across points of the bolt head or nut, $d_w = 40.0\ mm$

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

$= \dfrac{40.0}{4}$

$= 10.0\ mm$

Bolt tension area, $A_{b,t} = 352.5\ mm^2$

$k_2 = 0.9$

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

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

Column flange in bending (continued)

Design tension resistance of a bolt, $F_{t,Rd} = \dfrac{\left(\dfrac{k_2 \cdot f_{ub} \cdot A_{b,t}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{0.9 \cdot 800.0 \cdot 352.5}{1.25}\right)}{1000.0}$

$= 203.0\ kN$

$\gamma_{M0} = 1$

$M_{pl,1,Rd} = \dfrac{\left(\dfrac{0.25 \cdot l_{eff,1}(3,2) \cdot t_{f,c}^2 \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000000.0}$

$= \dfrac{\left(\dfrac{0.25 \cdot 323.0 \cdot 20.5^2 \cdot 265.0}{1}\right)}{1000000.0}$

$= 9.0\ kN \cdot m$

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

$= 9.0\ kN \cdot m$

$F_{T,1,Rd} = \left(\dfrac{(8 \cdot n_c - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{(2 \cdot m_c \cdot n_c - e_w \cdot (m_c + n_c))}\right) \cdot 1000.0$

$= \left(\dfrac{(8 \cdot 41.8 - 2 \cdot 10.0) \cdot 9.0}{(2 \cdot 33.4 \cdot 41.8 - 10.0 \cdot (33.4 + 41.8))}\right) \cdot 1000.0$

$= 1383.8\ kN$

$F_{T,2,Rd} = \left(\dfrac{\left(2 \cdot M_{pl,2,Rd} + \dfrac{n_c \cdot 2 \cdot 2 \cdot F_{t,Rd}}{1000.0}\right)}{(m_c + n_c)}\right) \cdot 1000.0$

$= \left(\dfrac{\left(2 \cdot 9.0 + \dfrac{41.8 \cdot 2 \cdot 2 \cdot 203.0}{1000.0}\right)}{(33.4 + 41.8)}\right) \cdot 1000.0$

$= 690.3\ kN$

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

$= 2 \cdot 2 \cdot 203.0$

$= 812.2\ kN$

Design resistance of column flange in bending, $F_{t,fc,Rd} = min\ (F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$= min\ (1383.8, 690.3, 812.2)$

Column flange in bending (continued)

$= 690.3\ kN$

Column web in tension

Effective width of column web in tension, $l_{eff,t,wc} = l_{eff,2}(3,2)$

$= 323.0\ mm$

Effective thickness of column web, $t_{wc,eff} = t_{w,c} + 0.5 \cdot t_{w,c} \cdot n_{dblr}$

$= 12.8 + 0.5 \cdot 12.8 \cdot 0$

$= 12.8\ mm$

Calculation of reduction factor for shear interaction

Transformation parameter, $\beta = min\left(\left|1 + \frac{M_{b2,Ed}}{M_{Ed}}\right|\right)$

$= min\left(\left|1 + \frac{-400.0}{400.0}\right|\right)$

$= 0$

Shear area of the column, $A_{vc} = A - 2 \cdot b_{f,c} \cdot t_{f,c} + (t_{w,c} + 2 \cdot r_c) \cdot t_{f,c}$

$= 13638.2 - 2 \cdot 258.8 \cdot 20.5 + (12.8 + 2 \cdot 12.7) \cdot 20.5$

$= 3810.5\ mm^2$

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{l_{eff,t,wc} \cdot t_{wc,eff}}{A_{vc}}\right)^2}}$$

$$= \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{323.0 \cdot 12.8}{3810.5}\right)^2}}$$

$= 0.628635$

$$\omega_2 = \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{l_{eff,t,wc} \cdot t_{wc,eff}}{A_{vc}}\right)^2}}$$

$$= \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{323.0 \cdot 12.8}{3810.5}\right)^2}}$$

$= 0.374715$

Reduction factor for shear interaction, $\omega = 1$

$\gamma_{M0} = 1$

Column web in tension (continued)

Design resistance of column web in tension, $F_{t,wc,Rd} = \dfrac{\left(\dfrac{\omega \cdot l_{eff,t,wc} \cdot t_{wc,eff} \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{1 \cdot 323.0 \cdot 12.8 \cdot 265.0}{1}\right)}{1000.0}$

$= 1095.6\ kN$

End plate in bending

Width across points of the bolt head or nut, $d_w = 40.0\ mm$

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

$= \dfrac{40.0}{4}$

$= 10.0\ mm$

Bolt tension area, $A_{b,t} = 352.5\ mm^2$

$k_2 = 0.9$

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

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

Design tension resistance of a bolt, $F_{t,Rd} = \dfrac{\left(\dfrac{k_2 \cdot f_{ub} \cdot A_{b,t}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{0.9 \cdot 800.0 \cdot 352.5}{1.25}\right)}{1000.0}$

$= 203.0\ kN$

$\gamma_{M0} = 1$

$M_{pl,1,Rd} = \dfrac{\left(\dfrac{0.25 \cdot l_{eff,1}(3,2) \cdot t_p^2 \cdot f_{y,p}}{\gamma_{M0}}\right)}{1000000.0}$

$= \dfrac{\left(\dfrac{0.25 \cdot 373.2 \cdot 25.0^2 \cdot 265.0}{1}\right)}{1000000.0}$

$= 15.5\ kN \cdot m$

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

$= 15.5\ kN \cdot m$

End plate in bending (continued)

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

$$= \left(\frac{(8 \cdot 48.2 - 2 \cdot 10.0) \cdot 15.5}{(2 \cdot 38.5 \cdot 48.2 - 10.0 \cdot (38.5 + 48.2))}\right) \cdot 1000.0$$

$$= 1983.5\ kN$$

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

$$= \left(\frac{\left(2 \cdot 15.5 + \frac{48.2 \cdot 2 \cdot 2 \cdot 203.0}{1000.0}\right)}{(38.5 + 48.2)}\right) \cdot 1000.0$$

$$= 807.6\ kN$$

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

$$= 2 \cdot 2 \cdot 203.0$$

$$= 812.2\ kN$$

Design resistance of end plate in bending, $F_{t,ep,Rd} = min\ (F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$$= min\ (1983.5, 807.6, 812.2)$$

$$= 807.6\ kN$$

Beam web in tension

Effective width of beam web in tension, $l_{eff,t,wb} = min\ (l_{eff,1}(3,2), l_{eff,2}(3,2))$

$$= min\ (373.2, 373.2)$$

$$= 373.2\ mm$$

$$\gamma_{M0} = 1$$

Design resistance of beam web in tension, $F_{t,wb,Rd} = \frac{\left(\frac{l_{eff,t,wb} \cdot t_{w,bt} \cdot f_{y,b}}{\gamma_{M0}}\right)}{1000.0}$

$$= \frac{\left(\frac{373.2 \cdot 10.1 \cdot 275.0}{1}\right)}{1000.0}$$

$$= 1036.7\ kN$$

$$F_{t,Rd,EL}(3,2) = F_{t,fc,Rd}$$

$$= 690.3\ kN$$

Effective design tension resistances for bolt-row group with bolt-rows 1-3

Column flange in bending

Width across points of the bolt head or nut, $d_w = 40.0\ mm$

$$e_w = \frac{d_w}{4}$$
$$= \frac{40.0}{4}$$
$$= 10.0\ mm$$

Bolt tension area, $A_{b,t} = 352.5\ mm^2$

$k_2 = 0.9$

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

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

Design tension resistance of a bolt, $F_{t,Rd} = \dfrac{\left(\dfrac{k_2 \cdot f_{ub} \cdot A_{b,t}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$$= \frac{\left(\frac{0.9 \cdot 800.0 \cdot 352.5}{1.25}\right)}{1000.0}$$

$$= 203.0\ kN$$

$\gamma_{M0} = 1$

$$M_{pl,1,Rd} = \frac{\left(\frac{0.25 \cdot l_{eff,1}(3,3) \cdot t_{f,c}^2 \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000000.0}$$

$$= \frac{\left(\frac{0.25 \cdot 423.0 \cdot 20.5^2 \cdot 265.0}{1}\right)}{1000000.0}$$

$$= 11.8\ kN \cdot m$$

$$M_{pl,2,Rd} = M_{pl,1,Rd}$$
$$= 11.8\ kN \cdot m$$

$$F_{T,1,Rd} = \left(\frac{(8 \cdot n_c - 2 \cdot e_w) \cdot M_{pl,1,Rd}}{(2 \cdot m_c \cdot n_c - e_w \cdot (m_c + n_c))}\right) \cdot 1000.0$$
$$= \left(\frac{(8 \cdot 41.8 - 2 \cdot 10.0) \cdot 11.8}{(2 \cdot 33.4 \cdot 41.8 - 10.0 \cdot (33.4 + 41.8))}\right) \cdot 1000.0$$
$$= 1812.3\ kN$$

$$F_{T,2,Rd} = \left(\frac{\left(2 \cdot M_{pl,2,Rd} + \frac{n_c \cdot 2 \cdot 3 \cdot F_{t,Rd}}{1000.0}\right)}{(m_c + n_c)}\right) \cdot 1000.0$$

Column flange in bending (continued)

$$= \left(\frac{\left(2 \cdot 11.8 + \frac{41.8 \cdot 2 \cdot 3 \cdot 203.0}{1000.0}\right)}{(33.4 + 41.8)} \right) \cdot 1000.0$$

$$= 989.9\ kN$$

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

$$= 2 \cdot 3 \cdot 203.0$$

$$= 1218.3\ kN$$

Design resistance of column flange in bending, $F_{t,fc,Rd} = min\ (F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$

$$= min\ (1812.3, 989.9, 1218.3)$$

$$= 989.9\ kN$$

Column web in tension

Effective width of column web in tension, $l_{eff,t,wc} = l_{eff,2}(3,3)$

$$= 423.0\ mm$$

Effective thickness of column web, $t_{wc,eff} = t_{w,c} + 0.5 \cdot t_{w,c} \cdot n_{dblr}$

$$= 12.8 + 0.5 \cdot 12.8 \cdot 0$$

$$= 12.8\ mm$$

Calculation of reduction factor for shear interaction

Transformation parameter, $\beta = min\ \left(\left| 1 + \frac{M_{b2,Ed}}{M_{Ed}} \right| \right)$

$$= min\ \left(\left| 1 + \frac{-400.0}{400.0} \right| \right)$$

$$= 0$$

Shear area of the column, $A_{vc} = A - 2 \cdot b_{f,c} \cdot t_{f,c} + (t_{w,c} + 2 \cdot r_c) \cdot t_{f,c}$

$$= 13638.2 - 2 \cdot 258.8 \cdot 20.5 + (12.8 + 2 \cdot 12.7) \cdot 20.5$$

$$= 3810.5\ mm^2$$

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{l_{eff,t,wc} \cdot t_{wc,eff}}{A_{vc}} \right)^2}}$$

$$= \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{423.0 \cdot 12.8}{3810.5} \right)^2}}$$

$$= 0.52524$$

Calculation of reduction factor for shear interaction (continued)

$$\omega_2 = \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{l_{eff,t,wc} \cdot t_{wc,eff}}{A_{vc}}\right)^2}}$$

$$= \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{423.0 \cdot 12.8}{3810.5}\right)^2}}$$

$= 0.294894$

Reduction factor for shear interaction, $\omega = 1$

$\gamma_{M0} = 1$

Design resistance of column web in tension, $F_{t,wc,Rd} = \frac{\left(\frac{\omega \cdot l_{eff,t,wc} \cdot t_{wc,eff} \cdot f_{y,c}}{\gamma_{M0}}\right)}{1000.0}$

$$= \frac{\left(\frac{1 \cdot 423.0 \cdot 12.8 \cdot 265.0}{1}\right)}{1000.0}$$

$= 1434.8\ kN$

$F_{t,Rd,EL}(3,3) = F_{t,fc,Rd}$

$= 989.9\ kN$

Effective design tension resistances for each bolt-row

$F_{t,Rd,row}(1,1) = F_{t,Rd,EL}(1,1)$

$= 376.9\ kN$

Effective design tension resistance of bolt-row 1, $F_{t1,Rd} = F_{t,Rd,row}(1,1)$

$= 376.9\ kN$

$F_{t,Rd,row}(2,1) = F_{t,Rd,EL}(2,1)$

$= 398.0\ kN$

$F_{t,Rd,row}(2,2) = F_{t,Rd,EL}(2,2)$

$= 697.7\ kN$

$F_{t,Rd,row}(2,2) = F_{t,Rd,row}(2,2) - F_{t1,Rd}$

$= 697.7 - 376.9$

$= 320.7\ kN$

Effective design tension resistance of bolt-row 2, $F_{t2,Rd} = F_{t,Rd,row}(2,1)$

$= 398.0\ kN$

Effective design tension resistance of bolt-row 2, $F_{t2,Rd} = F_{t,Rd,row}(2,2)$

$= 320.7\ kN$

Effective design tension resistances for each bolt-row (continued)

$F_{t,Rd,row}(3,1) = F_{t,Rd,EL}(3,1)$
$= 398.0\ kN$

$F_{t,Rd,row}(3,2) = F_{t,Rd,EL}(3,2)$
$= 690.3\ kN$

$F_{t,Rd,row}(3,2) = F_{t,Rd,row}(3,2) - F_{t2,Rd}$
$= 690.3 - 320.7$
$= 369.5\ kN$

$F_{t,Rd,row}(3,3) = F_{t,Rd,EL}(3,3)$
$= 989.9\ kN$

$F_{t,Rd,row}(3,3) = F_{t,Rd,row}(3,3) - F_{t2,Rd}$
$= 989.9 - 320.7$
$= 669.2\ kN$

$F_{t,Rd,row}(3,3) = F_{t,Rd,row}(3,3) - F_{t1,Rd}$
$= 669.2 - 376.9$
$= 292.2\ kN$

Effective design tension resistance of bolt-row 3, $F_{t3,Rd} = F_{t,Rd,row}(3,1)$
$= 398.0\ kN$

Effective design tension resistance of bolt-row 3, $F_{t3,Rd} = F_{t,Rd,row}(3,2)$
$= 369.5\ kN$

Effective design tension resistance of bolt-row 3, $F_{t3,Rd} = F_{t,Rd,row}(3,3)$
$= 292.2\ kN$

Bolt tension area, $A_{b,t} = 352.5\ mm^2$
$k_2 = 0.9$
Bolt ultimate tensile strength, $f_{ub} = 800.0\ MPa$
$\gamma_{M2,1\text{-}8} = 1.25$

Design tension resistance of a bolt, $F_{t,Rd} = \dfrac{\left(\dfrac{k_2 \cdot f_{ub} \cdot A_{b,t}}{\gamma_{M2,1\text{-}8}}\right)}{1000.0}$

$$= \frac{\left(\frac{0.9 \cdot 800.0 \cdot 352.5}{1.25}\right)}{1000.0}$$

$= 203.0\ kN$

Total tension design resistance, $F_{t,Rd,total} = 0.0\ kN$
Total tension design resistance, $F_{t,Rd,total} = F_{t,Rd,total} + F_{t1,Rd}$
$= 0.0 + 376.9$
$= 376.9\ kN$
Total tension design resistance, $F_{t,Rd,total} = F_{t,Rd,total} + F_{t2,Rd}$
$= 376.9 + 320.7$

Moment resistance of end plate connection (362). Reference Chapter 2 (continued)

$= 697.7\ kN$

Total tension design resistance, $F_{t,Rd,total} = F_{t,Rd,total} + F_{t3,Rd}$

$= 697.7 + 292.2$

$= 989.9\ kN$

Transformation parameter, $\beta = min\left(\left|1 + \frac{M_{b2,Ed}}{M_{Ed}}\right|\right)$

$= min\left(\left|1 + \frac{-400.0}{400.0}\right|\right)$

$= 0$

Design shear resistance of column web panel

Distance between the centrelines of stiffeners, $d_s = h_{b,overall} - t_{f,b}$

$= 533.1 - 15.6$

$= 517.5\ mm$

Shear area of the column, $A_{vc} = A - 2 \cdot b_{f,c} \cdot t_{f,c} + (t_{w,c} + 2 \cdot r_c) \cdot t_{f,c}$

$= 13638.2 - 2 \cdot 258.8 \cdot 20.5 + (12.8 + 2 \cdot 12.7) \cdot 20.5$

$= 3810.5\ mm^2$

$\gamma_{M0} = 1$

Design shear resistance of column web panel, $V_{wp,Rd} = \frac{\left(\frac{\left(\frac{0.9 \cdot f_{y,c} \cdot A_{vc}}{\sqrt{3}}\right)}{\gamma_{M0}}\right)}{1000.0}$

$= \frac{\left(\frac{\left(\frac{0.9 \cdot 265.0 \cdot 3810.5}{\sqrt{3}}\right)}{1}\right)}{1000.0}$

$= 524.7\ kN$

Design resistance of the column web in transverse compression

$s_p = 2 \cdot t_p$

$= 2 \cdot 25.0$

$= 50.0\ mm$

Effective width of column web in compression, $b_{eff,c,wc} = t_{f,bc} + 2 \cdot s_{f,c} + 5 \cdot (t_{f,c} + r_c) + s_p$

$= 15.6 + 2 \cdot 12.0 + 5 \cdot (20.5 + 12.7) + 50.0$

Design resistance of the column web in transverse compression (continued)

$= 255.6\ mm$

Effective thickness of column web, $t_{wc,eff} = t_{w,c} + 0.5 \cdot t_{w,c} \cdot 0$

$= 12.8 + 0.5 \cdot 12.8 \cdot 0$

$= 12.8\ mm$

Steel modulus of elasticity, $E = 210000.0\ MPa$

Clear depth of the column web, $d_{wc} = h_c - 2 \cdot (r_c + t_{f,c})$

$= 266.7 - 2 \cdot (12.7 + 20.5)$

$= 200.3\ mm$

Plate slenderness, $\lambda_p = 0.932 \cdot \sqrt{\left(\dfrac{b_{eff,c,wc} \cdot d_{wc} \cdot f_{y,c}}{E \cdot t_{wc,eff}^2}\right)}$

$= 0.932 \cdot \sqrt{\left(\dfrac{255.6 \cdot 200.3 \cdot 265.0}{210000.0 \cdot 12.8^2}\right)}$

$= 0.585248$

Reduction factor for plate buckling, $\rho = 1$

Calculation of reduction factor for shear interaction

Transformation parameter, $\beta = min\left(\left|1 + \dfrac{M_{b2,Ed}}{M_{Ed}}\right|\right)$

$= min\left(\left|1 + \dfrac{-400.0}{400.0}\right|\right)$

$= 0$

Shear area of the column, $A_{vc} = A - 2 \cdot b_{f,c} \cdot t_{f,c} + (t_{w,c} + 2 \cdot r_c) \cdot t_{f,c}$

$= 13638.2 - 2 \cdot 258.8 \cdot 20.5 + (12.8 + 2 \cdot 12.7) \cdot 20.5$

$= 3810.5\ mm^2$

$$\omega_1 = \frac{1}{\sqrt{1 + 1.3 \cdot \left(\dfrac{b_{eff,c,wc} \cdot t_{wc,eff}}{A_{vc}}\right)^2}}$$

$$= \frac{1}{\sqrt{1 + 1.3 \cdot \left(\dfrac{255.6 \cdot 12.8}{3810.5}\right)^2}}$$

$= 0.714589$

$$\omega_2 = \frac{1}{\sqrt{1 + 5.2 \cdot \left(\dfrac{b_{eff,c,wc} \cdot t_{wc,eff}}{A_{vc}}\right)^2}}$$

Calculation of reduction factor for shear interaction (continued)

$$= \frac{1}{\sqrt{1 + 5.2 \cdot \left(\frac{255.6 \cdot 12.8}{3810.5} \right)^2}}$$

$$= 0.454858$$

Reduction factor for shear interaction, $\omega = 1$

Web longitudinal compressive stress, $\sigma_{com,Ed} = \left(\frac{N_{Ed}}{\left(2 \cdot t_{f,c} \cdot b_{f,c} + t_{wc,eff} \cdot \left(h_c - 2 \cdot t_{f,c}\right)\right)} \right) \cdot 1000.0$

$$= \left(\frac{0.0}{\left(2 \cdot 20.5 \cdot 258.8 + 12.8 \cdot \left(266.7 - 2 \cdot 20.5\right)\right)} \right) \cdot 1000.0$$

$$= 0.0\ MPa$$

Reduction factor for web compressive stress, $k_{wc} = 1$

$\gamma_{M1} = 1$

$\gamma_{M0} = 1$

Design resistance of column web in compression, $F_{c,wc,Rd} = \frac{min \Bigg(\frac{\omega \cdot k_{wc} \cdot b_{eff,c,wc} \cdot t_{wc,eff} \cdot f_{y,c}}{\gamma_{M0}}, \frac{\omega \cdot k_{wc} \cdot \rho \cdot b_{eff,c,wc} \cdot t_{wc,eff}}{\gamma_{M1}}}{1000.0}$

$$= \frac{min \left(\frac{1 \cdot 1 \cdot 255.6 \cdot 12.8 \cdot 265.0}{1}, \frac{1 \cdot 1 \cdot 1 \cdot 255.6 \cdot 12.8 \cdot 265.0}{1} \right)}{1000.0}$$

$$= 867.0\ kN$$

Design resistance of beam flange and web in compression

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

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

Tension force, vertical component, $N_{v,t,Ed} = 0.0\ kN$

Compression force, vertical component, $N_{v,c,Ed} = 0.0\ kN$

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

Axial force, vertical component, $N_{v,Ed} = 0.0\ kN$

Root radius, $r = 12.7\ mm$

Yield stress, $f_y = 275.0\ MPa$

Flange thickness, $t_f = 15.6\ mm$

Beam section overall breadth, $b = 209.3\ mm$

Web thickness, $t_w = 10.1\ mm$

Beam section overall depth, $h = 533.1\ mm$

Plastic section modulus about the major axis, $W_{pl,y} = 2360087.4\ mm^3$

Minimum elastic section modulus, $W_{el,min} = 2071938.5\ mm^3$

Load Calc 59 (59)

Applied member moment, $M_{Ed} = 400.0\ kN \cdot m$

Design axial force, $N_{Ed} = max\ (0.0{,}0.0)$

$= 0.0\ kN$

Cross-sectional shear area, $A_v = 11737.8\ mm^2$

Flange width, $b = 209.3\ mm$

Flange thickness, $t_f = 15.6\ mm$

Root radius, $r_b = 12.7\ mm$

$\eta = 1$

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

$= 533.1 - 15.6 - 15.6$

$= 501.9\ mm$

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

$= max\ \left(11737.8 - 2 \cdot 209.3 \cdot 15.6 + \left(10.1 + 2 \cdot 12.7\right) \cdot 15.6, 1 \cdot 501.9 \cdot 10.1\right)$

$= 5761.4\ mm^2$

$\gamma_{M0} = 1$

Design plastic shear resistance, $V_{pl,Rd} = \dfrac{\left(\dfrac{f_y \cdot A_{v,w}}{\sqrt{3} \cdot \gamma_{M0}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{275.0 \cdot 5761.4}{\sqrt{3} \cdot 1}\right)}{1000.0}$

$= 914.8\ kN$

Gross section area, $A_g = b \cdot t_f \cdot 2 + \left(h - 2 \cdot t_f\right) \cdot t_w + 4 \cdot r^2 \cdot \left(1 - \dfrac{\pi}{4}\right)$

$= 209.3 \cdot 15.6 \cdot 2 + \left(533.1 - 2 \cdot 15.6\right) \cdot 10.1 + 4 \cdot 12.7^2 \cdot \left(1 - 0.785398\right)$

$= 11737.8\ mm^2$

$\gamma_{M0} = 1$

Design plastic normal resistance, $N_{pl,Rd} = \dfrac{\left(\dfrac{f_y \cdot A_g}{\gamma_{M0}}\right)}{1000.0}$

$= \dfrac{\left(\dfrac{275.0 \cdot 11737.8}{1}\right)}{1000.0}$

$= 3227.9\ kN$

Reduced yield stress due to shear, $f_{y,r} = f_y$

$= 275.0\ MPa$

$\gamma_{M0} = 1$

Load Calc 59 (59) (continued)

Design plastic moment resistance, $M_{pl,Rd} = \frac{\left(\frac{f_{y,r} \cdot W_{pl,y}}{\gamma_{M0}}\right)}{1000000.0}$

$= \frac{\left(\frac{275.0 \cdot 2360087.4}{1}\right)}{1000000.0}$

$= 649.0\ kN \cdot m$

$Shear\ capacity = M_{pl,Rd}$

$= 649.0\ kN \cdot m$

$Unity = \frac{|M_{Ed}|}{Shear\ capacity}$

$= \frac{|400.0|}{649.0}$

$= 0.616333$

Design moment resistance of the beam, $M_{c,Rd} = 649.0\ kN \cdot m$

Design resistance of beam flange and web in compression, $F_{c,fb,Rd} = \left(\frac{M_{c,Rd}}{(h_b - t_{f,b})}\right) \cdot 1000.0$

$= \left(\frac{649.0}{(533.1 - 15.6)}\right) \cdot 1000.0$

$= 1254.1\ kN$

Design compression resistance, $F_{c,Rd} = F_{c,fb,Rd}$

$= 1254.1\ kN$

Design compression resistance, $F_{c,Rd} = F_{c,wc,Rd}$

$= 867.0\ kN$

Design moment resistance with tension, $M_{j,Rd,t} = 0.0\ kN \cdot m$

Design moment resistance with compression, $M_{j,Rd,c} = 0.0\ kN \cdot m$

Effective design tension resistance of bolt-row 1 with tension force, $F_{t1,Rd,t} = F_{t1,Rd}$

$= 376.9\ kN$

Effective design tension resistance of bolt-row 1 with compress force, $F_{t1,Rd,c} = F_{t1,Rd}$

$= 376.9\ kN$

Effective design tension resistance of bolt-row 2 with tension force, $F_{t2,Rd,t} = F_{t2,Rd}$

$= 320.7\ kN$

Effective design tension resistance of bolt-row 2 with compress force, $F_{t2,Rd,c} = F_{t2,Rd}$

$= 320.7\ kN$

Effective design tension resistance of bolt-row 3 with tension force, $F_{t3,Rd,t} = F_{t3,Rd}$

$= 292.2\ kN$

Effective design tension resistance of bolt-row 3 with compress force, $F_{t3,Rd,c} = F_{t3,Rd}$

$= 292.2\ kN$

Design moment resistance with tension

The amount of tension resistance exceeds compression resistance, $F_{exceed} = F_{t,Rd,total} - N_{h,t,Ed} - F_{c,Rd}$

$= 989.9 - 0.0 - 867.0$

$= 122.9\ kN$

Reduce the tension row force to achieve equilibrium

Effective design tension resistance of bolt-row 3 with tension force, $F_{t3,Rd,t} = F_{t3,Rd,t} - F_{exceed}$

$= 292.2 - 122.9$

$= 169.3\ kN$

The amount of tension resistance exceeds compression resistance, $F_{exceed} = 0.0\ kN$

Design moment resistance with tension, $M_{j,Rd,t} = 0.0\ kN \cdot m$

Design moment resistance with tension, $M_{j,Rd,t} = M_{j,Rd,t} + \frac{F_{t1,Rd,t} \cdot h_1}{1000.0}$

$= 0.0 + \frac{376.9 \cdot 565.3}{1000.0}$

$= 213.1\ kN \cdot m$

Design moment resistance with tension, $M_{j,Rd,t} = M_{j,Rd,t} + \frac{F_{t2,Rd,t} \cdot h_2}{1000.0}$

$= 213.1 + \frac{320.7 \cdot 465.3}{1000.0}$

$= 362.3\ kN \cdot m$

Design moment resistance with tension, $M_{j,Rd,t} = M_{j,Rd,t} + \frac{F_{t3,Rd,t} \cdot h_3}{1000.0}$

$= 362.3 + \frac{169.3 \cdot 375.3}{1000.0}$

$= 425.9\ kN \cdot m$

Distance of the axial load from the center of compression, $h_N = \left(\frac{h_b}{2} + h_h\right) - \frac{t_{f,bc}}{2}$

$= \left(\frac{533.1}{2} + 0.0\right) - \frac{15.6}{2}$

$= 258.7\ mm$

Design moment resistance with tension, $M_{j,Rd,t} = M_{j,Rd,t} - \frac{N_{h,t,Ed} \cdot h_N}{1000.0}$

$= 425.9 - \frac{0.0 \cdot 258.7}{1000.0}$

$= 425.9\ kN \cdot m$

Design moment resistance with compression

The amount of tension resistance exceeds compression resistance, $F_{exceed} = \left(F_{t,Rd,total} + N_{h,c,Ed}\right) - F_{c,Rd}$

$= (989.9 + 0.0) - 867.0$

$= 122.9\ kN$

Reduce the tension row force to achieve equilibrium

Effective design tension resistance of bolt-row 3 with compress force, $F_{t3,Rd,c} = F_{t3,Rd,c} - F_{exceed}$

$= 292.2 - 122.9$

$= 169.3\ kN$

Reduce the tension row force to achieve equilibrium (continued)

The amount of tension resistance exceeds compression resistance, $F_{exceed} = 0.0\ kN$

Design moment resistance with compression, $M_{j,Rd,c} = 0.0\ kN \cdot m$

Design moment resistance with compression, $M_{j,Rd,c} = M_{j,Rd,c} + \frac{F_{t1,Rd,c} \cdot h_1}{1000.0}$

$= 0.0 + \frac{376.9 \cdot 565.3}{1000.0}$

$= 213.1\ kN \cdot m$

Design moment resistance with compression, $M_{j,Rd,c} = M_{j,Rd,c} + \frac{F_{t2,Rd,c} \cdot h_2}{1000.0}$

$= 213.1 + \frac{320.7 \cdot 465.3}{1000.0}$

$= 362.3\ kN \cdot m$

Design moment resistance with compression, $M_{j,Rd,c} = M_{j,Rd,c} + \frac{F_{t3,Rd,c} \cdot h_3}{1000.0}$

$= 362.3 + \frac{169.3 \cdot 375.3}{1000.0}$

$= 425.9\ kN \cdot m$

Distance of the axial load from the center of compression, $h_N = \left(\frac{h_b}{2} + h_h\right) - \frac{t_{f,bc}}{2}$

$= \left(\frac{533.1}{2} + 0.0\right) - \frac{15.6}{2}$

$= 258.7\ mm$

Design moment resistance with compression, $M_{j,Rd,c} = M_{j,Rd,c} + \frac{N_{h,c,Ed} \cdot h_N}{1000.0}$

$= 425.9 + \frac{0.0 \cdot 258.7}{1000.0}$

$= 425.9\ kN \cdot m$

Design moment resistance, $M_{j,Rd} = M_{j,Rd,t}$

$= 425.9\ kN \cdot m$

$Moment\ capacity = M_{j,Rd}$

$= 425.9\ kN \cdot m$

$Unity = \frac{|M_{Ed}|}{Moment\ capacity}$

$= \frac{|400.0|}{425.9}$

$= 0.939188$

$425.9\ kN \cdot m \geq (|400.0| = 400.0\ kN \cdot m)$ **(OK)**

$0.939 \leq 1$ **(OK)**

Results summary

Moment End Plate on right end of Beam B_17 [19]

Design calculation summary for member [19], right end

Desc. Ref.:	Calc. Num.	Unity Ratio	Resistance	Design Force	Green book ref.
1:	(362)	0.939	425.9 kN·m	-400.0 kN·m	Chapter 2
2:	(49)	0.040	498.1 kN	20.0 kN	Pg 27
3:	(2)	0.050	402.4 kN	20.0 kN	Pg 86
4:	(59)	0.616	649.0 kN·m	-400.0 kN·m	6.2, Part 1-1
6:	(110)	0.007	2668.2 kN	20.0 kN	Pg 22
7:	(110)	0.009	2187.9 kN	20.0 kN	Pg 22

Check number reference

1:	End plate moment
2:	Connection-bolt shear
3:	Supported beam-web shear
4:	Supported beam-moment strength
6:	Connection-bearing on plate
7:	Connection-bearing on support

Connection strength

	Unity ratio:
Shear:	0.050
Moment:	0.939

Notes and conclusions

- Column resisting moment, W_{pl}*F_y/γ_{M0} = 393.37 kN·m
- CONNECTION IS OK
    - Design resistance equals or exceeds design forces.
- Web weld size develops full strength of web according to Step 7 of the Green Book.
- Flange weld size develops full strength of flange according to Step 7 of the Green Book.

Moment End Plate on left end of Beam B_18 [20]

Design calculation summary for member [20], left end

Desc. Ref.:	Calc. Num.	Unity Ratio	Resistance	Design Force	Green book ref.
1:	(362)	0.939	425.9 kN·m	400.0 kN·m	Chapter 2
2:	(49)	0.040	498.1 kN	20.0 kN	Pg 27
3:	(2)	0.050	402.4 kN	20.0 kN	Pg 86
4:	(59)	0.616	649.0 kN·m	400.0 kN·m	6.2, Part 1-1
6:	(110)	0.007	2668.2 kN	20.0 kN	Pg 22
7:	(110)	0.009	2187.9 kN	20.0 kN	Pg 22

Check number reference

1:	End plate moment
2:	Connection-bolt shear
3:	Supported beam-web shear
4:	Supported beam-moment strength
6:	Connection-bearing on plate
7:	Connection-bearing on support

Connection strength

	Unity ratio:
Shear:	0.050
Moment:	0.939

Notes and conclusions

- Column resisting moment, $W_{pl}*F_y/\gamma_{M0}$ = 393.37 kN·m
- CONNECTION IS OK
  - Design resistance equals or exceeds design forces.
- Web weld size develops full strength of web according to Step 7 of the Green Book.
- Flange weld size develops full strength of flange according to Step 7 of the Green Book.

Additional notes

Example C.1 in P398 "Moment-Resisting Joints to Eurocode 3"