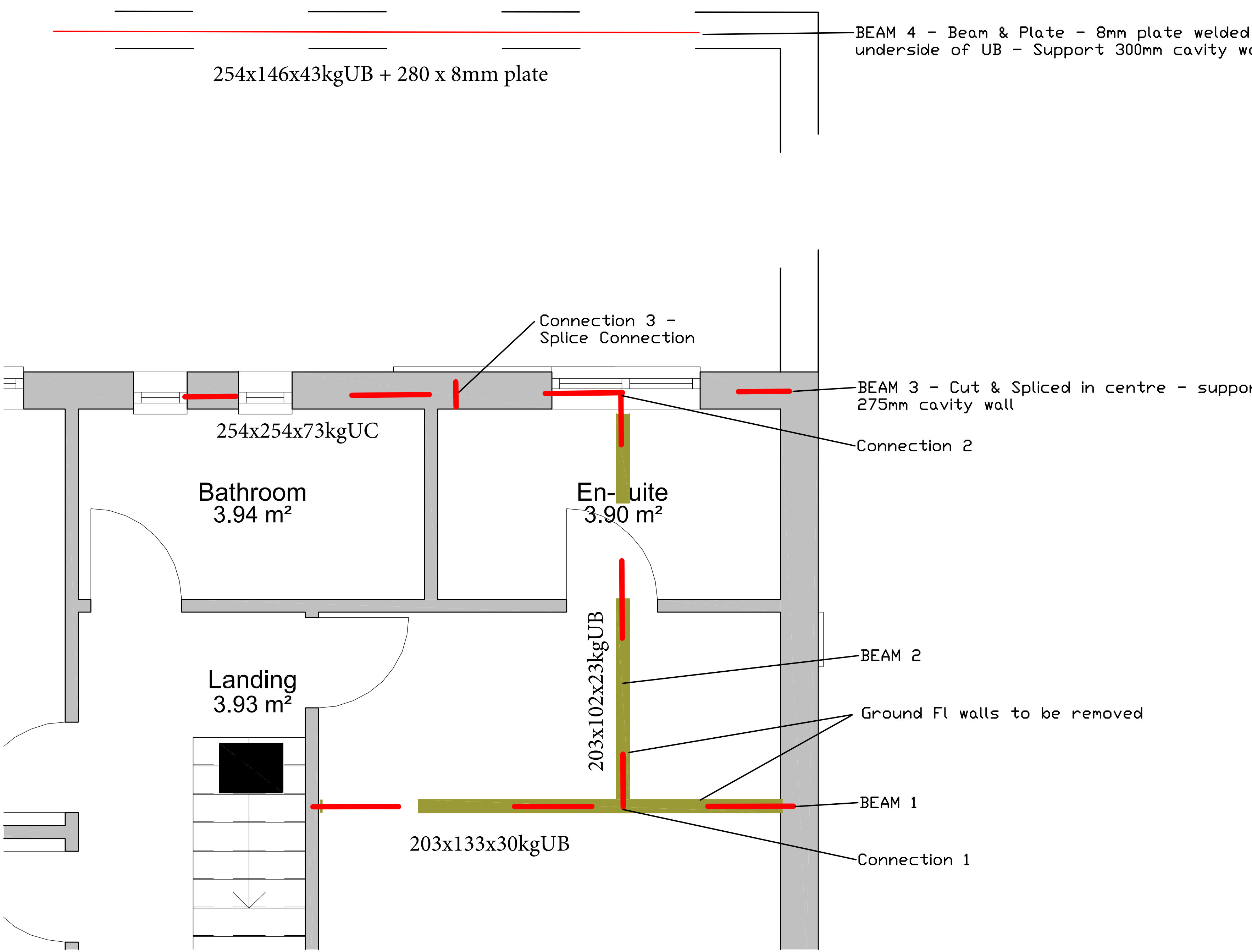


Beam Layout

Mr Connor McKitrick
1 Clover Drive
Freckleton
Preston PR4 1TG

Steel Beam Design

To Eurocode BS EN 1993-1-1/NA:2008

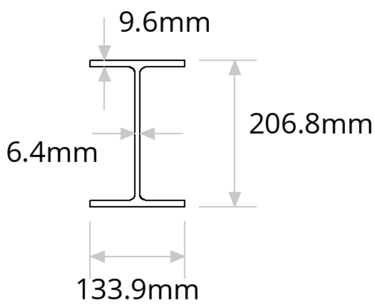
BEAM 1 - add 150mm bearing each end, total length 3880mm. Centre of connection point for BEAM 2 is 1224mm + 150mm = 1374mm from right.

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	**231** kN	**29.1** kN	**13** %	**OK**
Bending moment	**86.4** kNm	**26** kNm	**30** %	**OK**
Buckling	**62.8** kNm	**26** kNm	**41** %	**OK**
Total deflection	**17.9** mm	**3.9** mm	**22** %	**OK**
Deflection due to variable actions	**9.9** mm	**2.8** mm	**28** %	**OK**

Section details

Type	**Universal beam**
Section	**203 x 133 x 30 UB**
Steel grade	**S275**
Width	b = **134** mm
Depth	h = **207** mm
Web thickness	t_w = **6.4** mm
Flange thickness	t_f = **9.6** mm
Root radius	r = **7.6** mm
Mass per metre	w = **30** kg/m



Span and restraints

Effective span	L = **3,580** mm
Buckling length	L_{cr} = **3,580** mm

Deflection limits

Variable action deflection limit	Δ_Q = L / 360 = **9.94** mm
Total deflection limit	Δ_{G+Q} = L / 200 = **17.9** mm

Safety factors

Partial factor for permanent actions γ_G = **1.35**

Partial factor for variable actions γ_Q = **1.5**

Loading details

Self weight

Permanent action SW = w × 9.81 / 1000 = **0.294** kN/m

Load 1: UDL - Timber floor (domestic dwelling)

Permanent action G_1 = **0.6** kN/m² × **1.5** m = **0.9** kN/m

Variable action Q_1 = **1.5** kN/m² × **1.5** m = **2.25** kN/m

Load 2: UDL - Timber floor (domestic dwelling)

Permanent action G_2 = **0.6** kN/m² × **1.5** m = **0.9** kN/m

Variable action Q_2 = **1.5** kN/m² × **1.5** m = **2.25** kN/m

Load 3: UDL - Lightweight timber stud partitions, on floor plan

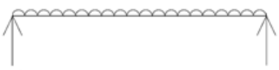
Permanent action G_3 = **0** kN/m² × **3.58** m = **0** kN/m

Variable action Q_3 = **0.25** kN/m² × **3.58** m = **0.895** kN/m

Load 4: UDL - Lightweight timber stud partitions, on floor plan

Permanent action G_4 = **0** kN/m² × **5** m = **0** kN/m

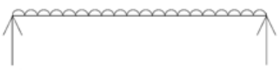
Variable action Q_4 = **0.25** kN/m² × **5** m = **1.25** kN/m



Load 5: UDL - Ceiling beneath sloping roof

Permanent action G_5 = **0.3** kN/m² × **1.5** m = **0.45** kN/m

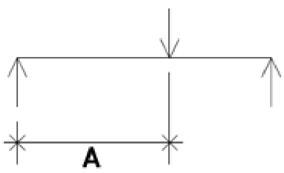
Variable action Q_5 = **0.25** kN/m² × **1.5** m = **0.375** kN/m



Load 6: UDL - Ceiling beneath sloping roof

Permanent action G_6 = **0.3** kN/m² × **1.5** m = **0.45** kN/m

Variable action Q_6 = **0.25** kN/m² × **1.5** m = **0.375** kN/m



Load 7: Point load - BEAM 2 Load

Distance to point load, A A = **2.4** m

Permanent action G_7 = **0.5** kN

Variable action Q_7 = **1.5** kN

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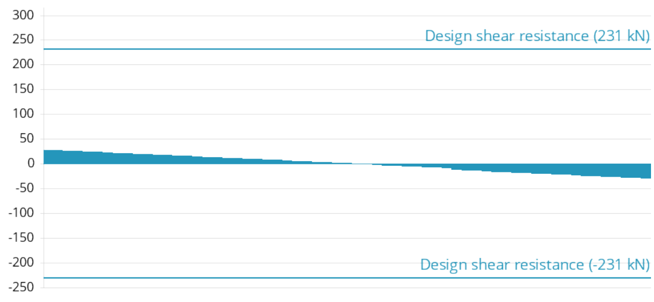
	Calcs for Connor McKitrich	Date 30 May 2023

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	**5.52** kN	**13.7** kN	**19.3** kN	**28.1** kN
Right reaction	**5.69** kN	**14.2** kN	**19.9** kN	**29.1** kN

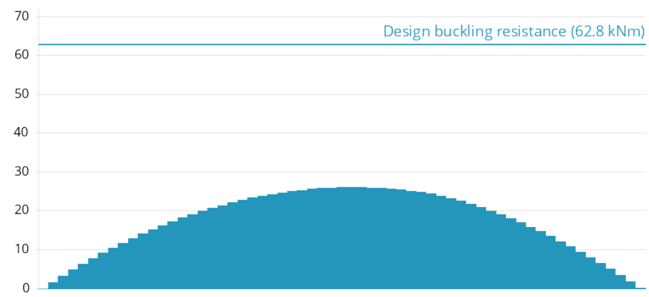
Design shear force



Shear force (kN)

Design shear force	V_{Ed} = **29.1** kN	
Design shear resistance	$V_{c,Rd}$ = **231** kN	
Utilisation	V_{Ed} / $V_{c,Rd}$ = **13** %	**OK**

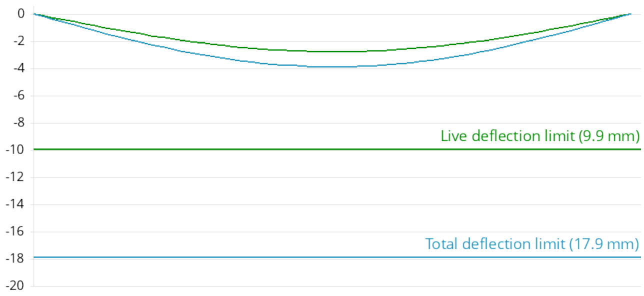
Design bending moment



Bending moments (kNm)

Design bending moment, major axis	M_{Ed} = **26** kNm	
Design resistance for bending	$M_{c,Rd}$ = **86.4** kNm	
Bending utilisation	M_{Ed} / $M_{c,Rd}$ = **30** %	**OK**
Design resistance for buckling	$M_{b,Rd}$ = **62.8** kNm	
Buckling utilisation	M_{Ed} / $M_{b,Rd}$ = **41** %	**OK**

Deflection



Live load deflection (green) and total load deflection (blue) in mm

Variable action deflection limit	Δ_Q = **9.9** mm	
Variable action deflection	δ_Q = **2.8** mm	**OK**
Total deflection limit	Δ_{G+Q} = **17.9** mm	
Total deflection	δ_{G+Q} = **3.9** mm	**OK**

Section properties

Elastic modulus - major axis, yy	W_{el} = **280** cm^3
Plastic modulus - major axis, yy	W_{pl} = **314** cm^3
Second moment of area - major axis, yy	I_y = **2,900** cm^4
Second moment of area - minor axis, zz	I_z = **385** cm^4
Warping constant	I_w = **0.0374** dm^6
Torsional constant	I_T = **10.3** cm^4
Area of section	A = **3,820** mm^2

Factors and design values of material coefficients (EN 1993-1-1:2005 and National Annex)

Young's modulus of elasticity	E = **210,000** N/mm^2	cl.3.2.6
Poisson's ratio in elastic stage	v = **0.3**	cl.3.2.6
Shear modulus	G_s = **81,000** N/mm^2	cl.3.2.6
Partial factor for resistance of cross-sections	γ_{M0} = **1**	cl.6.1(1)B / BS-EN NA
Partial factor for resistance to instability	γ_{M1} = **1**	cl.6.1(1)B / BS-EN NA
Factor for shear area	η = **1**	EN 1993-1-5:2006 cl.5.1(2) / BS-EN NA
Limiting non dimensional slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.4**	cl.6.3.2.3(1) / BS-EN NA
Beta factor for buckling reduction factor calculation	β = **0.75**	cl.6.3.2.3(1) / BS-EN NA

Yield strength

Nominal yield strength for S275 grade and nominal section thickness 9.60 mm	f_y = **275** N/mm^2	Tata blue book

Section classification (EN 1993-1-1:2005 cl.5.5)

Epsilon	ε = **0.924**	EN 1993-1-1:2005 Table 5.2
Flange ratio for local buckling	c_f / t_f = **5.85**	
Flange ratio limit for class 1	9 ε = **8.32**	Table 5.2 (sheet 2 of 3)
Flange class	$Class_f$ = **1**	
Web ratio for local buckling	c_w / t_w = **26.9**	
Web ratio limit for class 1	72 ε = **66.6**	Table 5.2 (sheet 1 of 3)

Web class	$Class_w$ = **1**	
Section class	Class = **1**	

Shear resistance (EN 1993-1-1:2005 cl.6.2.6)

Height of web	h_w = **188** mm	
Shear area for I and H sections	A_v = **1,460** mm^2	cl.6.2.6 (3)
Design shear resistance	$V_{pl,Rd}$ = **231** kN	eq (6.18)

Shear buckling (EN 1993-1-5:2006 cl.5)

The shear buckling resistance for webs should be verified according to Section 5 of EN 1993-1-5 if $(h_w / t_w) > (72\ \varepsilon / \eta)$

Web ratio for shear buckling	h_w / t_w = **29.3**	EN 1993-1-5:2006 cl.5.1 (2)
Shear buckling limit	$72\ \varepsilon / \eta$ = **66.6**	EN 1993-1-5:2006 cl.5.1 (2)

$(h_w / t_w) <= (72\ \varepsilon / \eta)$ therefore shear buckling calculation not required

Bending resistance (EN 1993-1-1:2005 cl.6.2.5)

The shear force (29 kN) is less than half of the plastic shear resistance (231 kN / 2 = 116 kN), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus	W_{pl} = **314,000** mm^3	
Design bending resistance	$M_{c,Rd}$ = **86.4** kNm	eq (6.13)

Design buckling resistance (EN 1993-1-1:2005 cl.6.3.2)

C1 factor	C1 = **1**	
Shear modulus of elasticity	G_s = **81,000** N/mm^2	cl.3.2.6 (1)
Buckling length	L_{cr} = **3,580** mm	
Critical buckling moment	M_{CR} = **94.7** kNm	NCCI SN003b-EN-EU
Class 1 section, therefore use plastic modulus	W_{pl} = **314,000** mm^3	cl.6.3.2.1(3)
Non-dimensional slenderness ratio	$\bar{\lambda}_{LT}$ = **0.955**	cl.6.3.2.2 (1)
Depth to width ratio for buckling curve	h / b = **1.54**	
Buckling curve for h / b ratio	Buckling curve = **b**	Table 6.5 / BS-EN NA
Imperfection factor for buckling curve b	α_{LT} = **0.34**	Table 6.3 / BS-EN NA
Intermediate factor for reduction factor calculation	Φ_{LT} = **0.936**	cl.6.3.2.3 (1)
Buckling reduction factor	χ_{LT} = **0.727**	eq (6.57)
Correction factor for moment distribution	k_c = **1**	Table 6.6
Moment distribution modification factor	f = **1**	cl.6.3.2.3 (2)
Modified buckling reduction factor	$\chi_{LT,mod}$ = **0.727**	eq (6.58)
Design buckling resistance	$M_{b,Rd}$ = **62.8** kNm	eq (6.55)

Notes

C1 value conservatively taken as 1.0

Ends of beam are to be laterally restrained. Ends of beams can be laterally restrained using one of the following methods;

1) End of beam built into masonry wall.

2) End of beam fixed to a masonry wall.
3) End of beam fixed to a column or a beam.

The designer is to ensure that the proposed detail adequately ensures that the end of the beam is laterally restrained.

No allowance has been made for destabilising loads which are outside the scope of these calculations (Destabilising loads would not normally occur in a traditional masonry structure)

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	Calcs for Connor McKitrich	Date 30 May 2023

Steel Beam Design

To Eurocode BS EN 1993-1-1/NA:2008

BEAM 2 - 3206mm total length including its connections to centre lines of BEAMS 1 & 3. BEAM 2 to be located underside of floor joists, as BEAM 1.

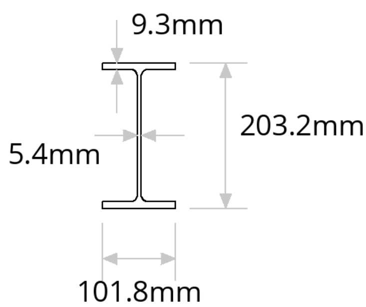
Alturnative, BEAM 2 can be placed higher in floor cavity and rest on top of BEAMS 1 & 3. BEAM 2 then needs to be longer with no connections 3506mm.

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	**197** kN	**2.41** kN	**1 %**	**OK**
Bending moment	**64.4** kNm	**1.93** kNm	**3 %**	**OK**
Buckling	**41.4** kNm	**1.93** kNm	**5 %**	**OK**
Total deflection	**16** mm	**0.3** mm	**2 %**	**OK**
Deflection due to variable actions	**8.9** mm	**0.2** mm	**3 %**	**OK**

Section details

Type	**Universal beam**
Section	**203 x 102 x 23 UB**
Steel grade	**S275**
Width	b = **102** mm
Depth	h = **203** mm
Web thickness	t_w = **5.4** mm
Flange thickness	t_f = **9.3** mm
Root radius	r = **7.6** mm
Mass per metre	w = **23.1** kg/m



Span and restraints

Effective span	L = **3,210** mm
Buckling length	L_{cr} = **3,210** mm

Deflection limits

Variable action deflection limit	Δ_Q = L / 360 = **8.91** mm
Total deflection limit	Δ_{G+Q} = L / 200 = **16** mm

Safety factors

Partial factor for permanent actions γ_G = **1.35**

Partial factor for variable actions γ_Q = **1.5**

Loading details

Self weight

Permanent action SW = w × 9.81 / 1000 = **0.227** kN/m

Load 1: UDL - Lightweight timber stud partitions, on floor plan

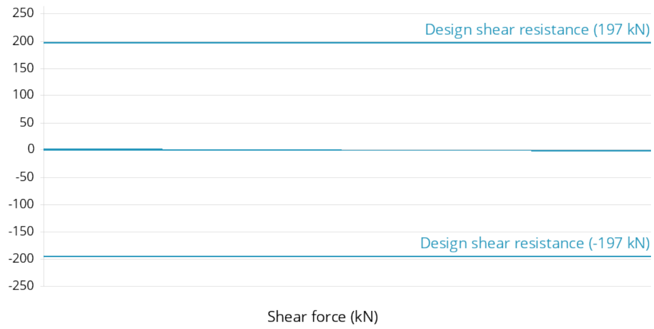
Permanent action G_1 = **0** kN/m² × **3.2** m = **0** kN/m

Variable action Q_1 = **0.25** kN/m² × **3.2** m = **0.8** kN/m

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	**0.363** kN	**1.28** kN	**1.65** kN	**2.41** kN
Right reaction	**0.363** kN	**1.28** kN	**1.65** kN	**2.41** kN

Design shear force

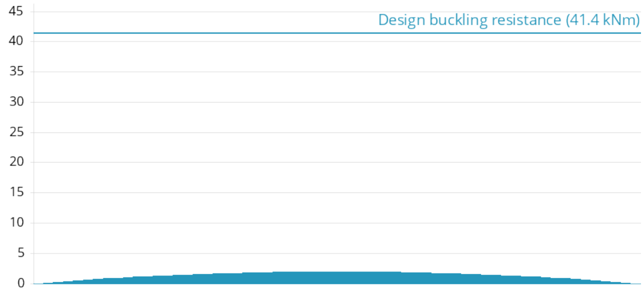


Design shear force V_{Ed} = **2.41** kN

Design shear resistance $V_{c,Rd}$ = **197** kN

Utilisation V_{Ed} / $V_{c,Rd}$ = **1** % **OK**

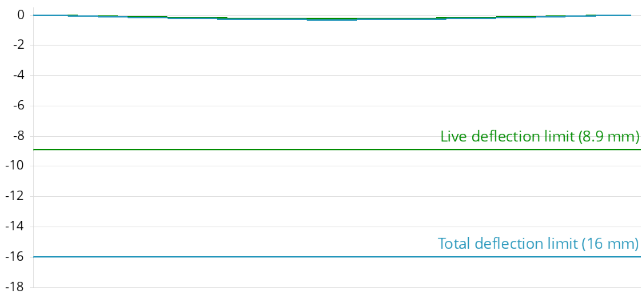
Design bending moment



Bending moments (kNm)

Design bending moment, major axis	M_{Ed} = **1.93** kNm	
Design resistance for bending	$M_{c,Rd}$ = **64.4** kNm	
Bending utilisation	M_{Ed} / $M_{c,Rd}$ = **3** %	**OK**
Design resistance for buckling	$M_{b,Rd}$ = **41.4** kNm	
Buckling utilisation	M_{Ed} / $M_{b,Rd}$ = **5** %	**OK**

Deflection



Live load deflection (green) and total load deflection (blue) in mm

Variable action deflection limit	Δ_Q = **8.9** mm	
Variable action deflection	δ_Q = **0.2** mm	**OK**
Total deflection limit	Δ_{G+Q} = **16** mm	
Total deflection	δ_{G+Q} = **0.3** mm	**OK**

Section properties

Elastic modulus - major axis, yy	W_{el} = **207** cm^3
Plastic modulus - major axis, yy	W_{pl} = **234** cm^3
Second moment of area - major axis, yy	I_y = **2,100** cm^4
Second moment of area - minor axis, zz	I_z = **164** cm^4
Warping constant	I_w = **0.0154** dm^6
Torsional constant	I_T = **7.02** cm^4
Area of section	A = **2,940** mm^2

Factors and design values of material coefficients (EN 1993-1-1:2005 and National Annex)

Young's modulus of elasticity	E = **210,000** N/mm^2	cl.3.2.6
Poisson's ratio in elastic stage	ν = **0.3**	cl.3.2.6
Shear modulus	G_s = **81,000** N/mm^2	cl.3.2.6
Partial factor for resistance of cross-sections	γ_{M0} = **1**	cl.6.1(1)B / BS-EN NA
Partial factor for resistance to instability	γ_{M1} = **1**	cl.6.1(1)B / BS-EN NA
Factor for shear area	η = **1**	EN 1993-1-5:2006 cl.5.1(2) / BS-EN NA
Limiting non dimensional slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.4**	cl.6.3.2.3(1) / BS-EN NA

Beta factor for buckling reduction factor calculation	β = **0.75**	cl.6.3.2.3(1) / BS-EN NA

Yield strength

Nominal yield strength for S275 grade and nominal section thickness 9.30 mm	f_y = **275** N/mm²	Tata blue book

Section classification (EN 1993-1-1:2005 cl.5.5)

Epsilon	ε = **0.924**	EN 1993-1-1:2005 Table 5.2
Flange ratio for local buckling	c_f / t_f = **4.37**	
Flange ratio limit for class 1	$9\ \varepsilon$ = **8.32**	Table 5.2 (sheet 2 of 3)
Flange class	$Class_f$ = **1**	
Web ratio for local buckling	c_w / t_w = **31.4**	
Web ratio limit for class 1	$72\ \varepsilon$ = **66.6**	Table 5.2 (sheet 1 of 3)
Web class	$Class_w$ = **1**	
Section class	Class = **1**	

Shear resistance (EN 1993-1-1:2005 cl.6.2.6)

Height of web	h_w = **185** mm	
Shear area for I and H sections	A_v = **1,240** mm²	cl.6.2.6 (3)
Design shear resistance	$V_{pl,Rd}$ = **197** kN	eq (6.18)

Shear buckling (EN 1993-1-5:2006 cl.5)

The shear buckling resistance for webs should be verified according to Section 5 of EN 1993-1-5 if $(h_w / t_w) > (72\ \varepsilon / \eta)$

Web ratio for shear buckling	h_w / t_w = **34.2**	EN 1993-1-5:2006 cl.5.1 (2)
Shear buckling limit	$72\ \varepsilon / \eta$ = **66.6**	EN 1993-1-5:2006 cl.5.1 (2)

$(h_w / t_w) <= (72\ \varepsilon / \eta)$ therefore shear buckling calculation not required

Bending resistance (EN 1993-1-1:2005 cl.6.2.5)

The shear force (2 kN) is less than half of the plastic shear resistance (197 kN / 2 = 98 kN), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus	W_{pl} = **234,000** mm³	
Design bending resistance	$M_{c,Rd}$ = **64.4** kNm	eq (6.13)

Design buckling resistance (EN 1993-1-1:2005 cl.6.3.2)

C1 factor	C1 = **1**	
Shear modulus of elasticity	G_s = **81,000** N/mm²	cl.3.2.6 (1)
Buckling length	L_{cr} = **3,210** mm	
Critical buckling moment	M_{CR} = **53.9** kNm	NCCI SN003b-EN-EU
Class 1 section, therefore use plastic modulus	W_{pl} = **234,000** mm³	cl.6.3.2.1(3)
Non-dimensional slenderness ratio	$\bar{\lambda}_{LT}$ = **1.09**	cl.6.3.2.2 (1)
Depth to width ratio for buckling curve	h / b = **2**	
Buckling curve for h / b ratio	Buckling curve = **b**	Table 6.5 / BS-EN NA
Imperfection factor for buckling curve b	α_{LT} = **0.34**	Table 6.3 / BS-EN NA

Intermediate factor for reduction factor calculation	ϕ_{LT} = **1.07**	cl.6.3.2.3 (1)
Buckling reduction factor	χ_{LT} = **0.643**	eq (6.57)
Correction factor for moment distribution	k_c = **1**	Table 6.6
Moment distribution modification factor	f = **1**	cl.6.3.2.3 (2)
Modified buckling reduction factor	$\chi_{LT,mod}$ = **0.643**	eq (6.58)
Design buckling resistance	$M_{b,Rd}$ = **41.4** kNm	eq (6.55)

Notes

C1 value conservatively taken as 1.0

Ends of beam are to be laterally restrained. Ends of beams can be laterally restrained using one of the following methods;

1) End of beam built into masonry wall.
2) End of beam fixed to a masonry wall.
3) End of beam fixed to a column or a beam.

The designer is to ensure that the proposed detail adequately ensures that the end of the beam is laterally restrained.

No allowance has been made for destabilising loads which are outside the scope of these calculations (Destabilising loads would not normally occur in a traditional masonry structure)

PlanningApplications.com Summer House, Upper Court Road, SURREY. CR3 7BF 0203 294 9477 www.planningapplications.com support@planningapplications.com	Project BEAM 3 (with Cut & Splice in centre)	Project ref 2023-7459
	Calcs for Connor McKitrich	Date 30 May 2023

Steel Beam Design

To Eurocode BS EN 1993-1-1/NA:2008

To support rear elevation cavity wall (275-300mm width) BEAM 3 254x254x73kgUC. Total beam length 4555mm + 150mm + 150mm = 4855mm.

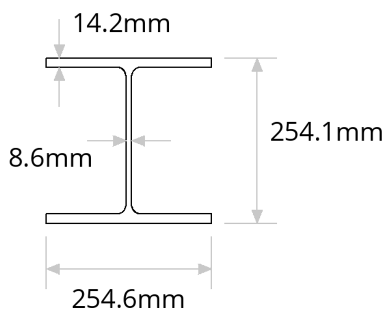
Point load to carry BEAM 2, 1224mm + 150mm = 1374mm connection centre in from right (Connection 2).

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	**407** kN	**77.2** kN	**19** %	**OK**
Bending moment	**273** kNm	**87.2** kNm	**32** %	**OK**
Buckling	**243** kNm	**87.2** kNm	**36** %	**OK**
Total deflection	**22.8** mm	**5.6** mm	**25** %	**OK**
Deflection due to variable actions	**12.7** mm	**1.9** mm	**15** %	**OK**

Section details

Type	**Universal column**
Section	**254 x 254 x 73 UC**
Steel grade	**S275**
Width	b = **255** mm
Depth	h = **254** mm
Web thickness	t_w = **8.6** mm
Flange thickness	t_f = **14.2** mm
Root radius	r = **12.7** mm
Mass per metre	w = **73.1** kg/m



Span and restraints

Effective span	L = **4,560** mm
Buckling length	L_{cr} = **4,560** mm

Deflection limits

Variable action deflection limit	Δ_Q = L / 360 = **12.6** mm
Total deflection limit	Δ_{G+Q} = L / 200 = **22.8** mm

Safety factors

Partial factor for permanent actions γ_G = **1.35**

Partial factor for variable actions γ_Q = **1.5**

Loading details

Self weight

Permanent action SW = w × 9.81 / 1000 = **0.717** kN/m

Load 1: UDL - 100mm Lightweight blockwork + Plaster or render on ONE side

Permanent action G_1 = **1** kN/m² × **2.5** m = **2.5** kN/m

Variable action Q_1 = **0** kN/m² × **2.5** m = **0** kN/m

Load 2: UDL - 102.5mm Brickwork + Plaster or render on ONE side

Permanent action G_2 = **2.25** kN/m² × **2.5** m = **5.62** kN/m

Variable action Q_2 = **0** kN/m² × **2.5** m = **0** kN/m

Load 3: UDL - Sloping roof, 0° to 30°

Permanent action G_3 = **1.15** kN/m² × **3** m = **3.45** kN/m

Variable action Q_3 = **0.75** kN/m² × **3** m = **2.25** kN/m

Load 4: UDL - Sloping roof, 0° to 30°

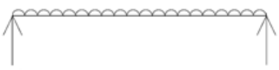
Permanent action G_4 = **1.15** kN/m² × **2** m = **2.3** kN/m

Variable action Q_4 = **0.75** kN/m² × **2** m = **1.5** kN/m

Load 5: UDL - Timber floor (domestic dwelling)

Permanent action G_5 = **0.6** kN/m² × **1.5** m = **0.9** kN/m

Variable action Q_5 = **1.5** kN/m² × **1.5** m = **2.25** kN/m



Load 6: UDL - Ceiling beneath sloping roof

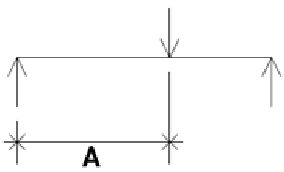
Permanent action G_6 = **0.3** kN/m² × **1.5** m = **0.45** kN/m

Variable action Q_6 = **0.25** kN/m² × **1.5** m = **0.375** kN/m

Load 7: UDL - Lightweight timber stud partitions, on floor plan

Permanent action G_7 = **0** kN/m² × **5** m = **0** kN/m

Variable action Q_7 = **0.25** kN/m² × **5** m = **1.25** kN/m



Load 8: Point load - BEAM 2 Load

Distance to point load, A A = **3.35** m

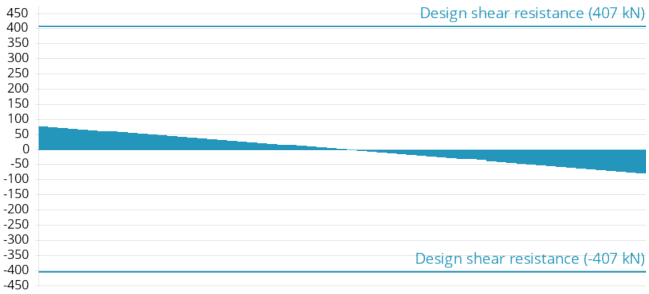
Permanent action G_8 = **0.5** kN

Variable action Q_8 = **1.5** kN

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	**36.4** kN	**17.8** kN	**54.2** kN	**75.8** kN
Right reaction	**36.7** kN	**18.5** kN	**55.1** kN	**77.2** kN

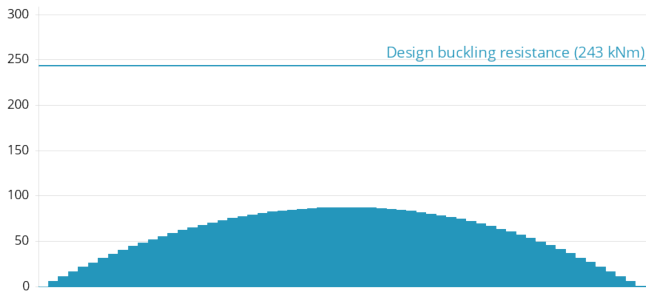
Design shear force



Shear force (kN)

Design shear force	V_{Ed} = **77.2** kN	
Design shear resistance	$V_{c,Rd}$ = **407** kN	
Utilisation	V_{Ed} / $V_{c,Rd}$ = **19** %	**OK**

Design bending moment



Bending moments (kNm)

Design bending moment, major axis	M_{Ed} = **87.2** kNm	
Design resistance for bending	$M_{c,Rd}$ = **273** kNm	
Bending utilisation	M_{Ed} / $M_{c,Rd}$ = **32** %	**OK**
Design resistance for buckling	$M_{b,Rd}$ = **243** kNm	
Buckling utilisation	M_{Ed} / $M_{b,Rd}$ = **36** %	**OK**

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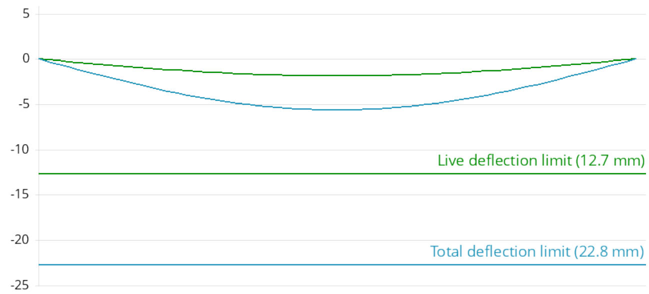
	Calcs for Connor McKitrich	Date 30 May 2023

Deflection



Live load deflection (green) and total load deflection (blue) in mm

Variable action deflection limit	Δ_Q = **12.7** mm	
Variable action deflection	δ_Q = **1.9** mm	**OK**
Total deflection limit	Δ_{G+Q} = **22.8** mm	
Total deflection	δ_{G+Q} = **5.6** mm	**OK**

Section properties

Elastic modulus - major axis, yy	W_{el} = **898** cm^3	
Plastic modulus - major axis, yy	W_{pl} = **992** cm^3	
Second moment of area - major axis, yy	I_y = **11,400** cm^4	
Second moment of area - minor axis, zz	I_z = **3,910** cm^4	
Warping constant	I_w = **0.562** dm^6	
Torsional constant	I_T = **57.6** cm^4	
Area of section	A = **9,310** mm^2	

Factors and design values of material coefficients (EN 1993-1-1:2005 and National Annex)

Young's modulus of elasticity	E = **210,000** N/mm^2	cl.3.2.6
Poisson's ratio in elastic stage	ν = **0.3**	cl.3.2.6
Shear modulus	G_s = **81,000** N/mm^2	cl.3.2.6
Partial factor for resistance of cross-sections	γ_{M0} = **1**	cl.6.1(1)B / BS-EN NA
Partial factor for resistance to instability	γ_{M1} = **1**	cl.6.1(1)B / BS-EN NA
Factor for shear area	η = **1**	EN 1993-1-5:2006 cl.5.1(2) / BS-EN NA
Limiting non dimensional slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.4**	cl.6.3.2.3(1) / BS-EN NA
Beta factor for buckling reduction factor calculation	β = **0.75**	cl.6.3.2.3(1) / BS-EN NA

Yield strength

Nominal yield strength for S275 grade and nominal section thickness 14.20 mm	f_y = **275** N/mm^2	Tata blue book

Section classification (EN 1993-1-1:2005 cl.5.5)

Epsilon	ε = **0.924**	EN 1993-1-1:2005 Table 5.2
Flange ratio for local buckling	c_f / t_f = **7.77**	
Flange ratio limit for class 1	9 ε = **8.32**	Table 5.2 (sheet 2 of 3)
Flange class	$Class_f$ = **1**	
Web ratio for local buckling	c_w / t_w = **23.3**	
Web ratio limit for class 1	72 ε = **66.6**	Table 5.2 (sheet 1 of 3)

Web class	$Class_w$ = **1**	
Section class	Class = **1**	

Shear resistance (EN 1993-1-1:2005 cl.6.2.6)

Height of web	h_w = **226** mm	
Shear area for I and H sections	A_v = **2,560** mm^2	cl.6.2.6 (3)
Design shear resistance	$V_{pl,Rd}$ = **407** kN	eq (6.18)

Shear buckling (EN 1993-1-5:2006 cl.5)

The shear buckling resistance for webs should be verified according to Section 5 of EN 1993-1-5 if (h_w / t_w) > (72 ε / η)

Web ratio for shear buckling	h_w / t_w = **26.2**	EN 1993-1-5:2006 cl.5.1 (2)
Shear buckling limit	72 ε / η = **66.6**	EN 1993-1-5:2006 cl.5.1 (2)

(h_w / t_w) <= (72 ε / η) therefore shear buckling calculation not required

Bending resistance (EN 1993-1-1:2005 cl.6.2.5)

The shear force (77 kN) is less than half of the plastic shear resistance (407 kN / 2 = 203 kN), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus	W_{pl} = **992,000** mm^3	
Design bending resistance	$M_{c,Rd}$ = **273** kNm	eq (6.13)

Design buckling resistance (EN 1993-1-1:2005 cl.6.3.2)

C1 factor	C1 = **1**	
Shear modulus of elasticity	G_s = **81,000** N/mm^2	cl.3.2.6 (1)
Buckling length	L_{cr} = **4,560** mm	
Critical buckling moment	M_{CR} = **634** kNm	NCCI SN003b-EN-EU
Class 1 section, therefore use plastic modulus	W_{pl} = **992,000** mm^3	cl.6.3.2.1(3)
Non-dimensional slenderness ratio	$\bar{\lambda}_{LT}$ = **0.656**	cl.6.3.2.2 (1)
Depth to width ratio for buckling curve	h / b = **0.998**	
Buckling curve for h / b ratio	Buckling curve = **b**	Table 6.5 / BS-EN NA
Imperfection factor for buckling curve b	α_{LT} = **0.34**	Table 6.3 / BS-EN NA
Intermediate factor for reduction factor calculation	Φ_{LT} = **0.705**	cl.6.3.2.3 (1)
Buckling reduction factor	χ_{LT} = **0.891**	eq (6.57)
Correction factor for moment distribution	k_c = **1**	Table 6.6
Moment distribution modification factor	f = **1**	cl.6.3.2.3 (2)
Modified buckling reduction factor	$\chi_{LT,mod}$ = **0.891**	eq (6.58)
Design buckling resistance	$M_{b,Rd}$ = **243** kNm	eq (6.55)

Notes

C1 value conservatively taken as 1.0

Ends of beam are to be laterally restrained. Ends of beams can be laterally restrained using one of the following methods;

1) End of beam built into masonry wall.

2) End of beam fixed to a masonry wall.
3) End of beam fixed to a column or a beam.

The designer is to ensure that the proposed detail adequately ensures that the end of the beam is laterally restrained.

No allowance has been made for destabilising loads which are outside the scope of these calculations (Destabilising loads would not normally occur in a traditional masonry structure)

Steel Beam Design

To Eurocode BS EN 1993-1-1/NA:2008

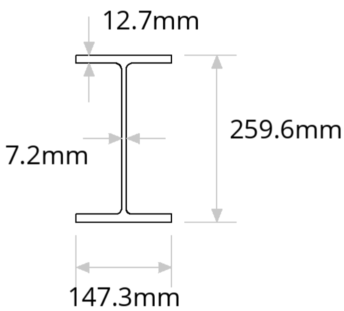
BEAM 4 - with 280mm x 8mm plate with 6mm fillet weld full length both sides. See additional Beam details with plate. Beam length 5000mm + 150mm + 150mm = 5300mm total length (150mm bearing each end).

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	**321** kN	**28.1** kN	**9 %**	**OK**
Bending moment	**156** kNm	**35.1** kNm	**23 %**	**OK**
Buckling	**98** kNm	**35.1** kNm	**36 %**	**OK**
Total deflection	**25** mm	**4.8** mm	**19 %**	**OK**
Deflection due to variable actions	**13.9** mm	**1.3** mm	**10 %**	**OK**

Section details

Type	**Universal beam**
Section	**254 x 146 x 43 UB**
Steel grade	**S275**
Width	b = **147** mm
Depth	h = **260** mm
Web thickness	t_w = **7.2** mm
Flange thickness	t_f = **12.7** mm
Root radius	r = **7.6** mm
Mass per metre	w = **43** kg/m



Span and restraints

Effective span	L = **5,000** mm
Buckling length	L_{cr} = **5,000** mm

Deflection limits

Variable action deflection limit	Δ_Q = L / 360 = **13.9** mm
Total deflection limit	Δ_{G+Q} = L / 200 = **25** mm

Safety factors

Partial factor for permanent actions γ_G = **1.35**

Partial factor for variable actions γ_Q = **1.5**

Loading details

Self weight

Permanent action SW = w × 9.81 / 1000 = **0.422** kN/m

Load 1: UDL - 100mm Lightweight blockwork + Plaster or render on ONE side

Permanent action G_1 = **1** kN/m^2 × **0.6** m = **0.6** kN/m

Variable action Q_1 = **0** kN/m^2 × **0.6** m = **0** kN/m

Load 2: UDL - 102.5mm Brickwork + Plaster or render on ONE side

Permanent action G_2 = **2.25** kN/m^2 × **0.6** m = **1.35** kN/m

Variable action Q_2 = **0** kN/m^2 × **0.6** m = **0** kN/m

Load 3: UDL - Sloping roof, 0° to 30°

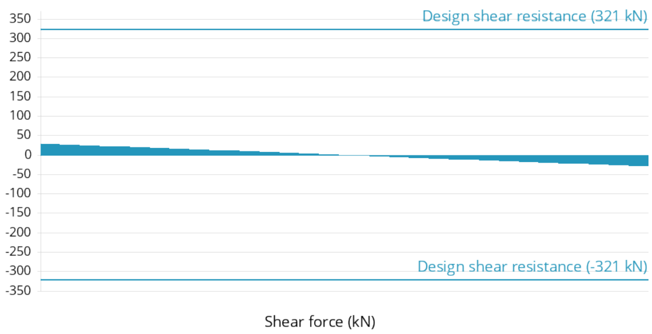
Permanent action G_3 = **1.15** kN/m^2 × **3** m = **3.45** kN/m

Variable action Q_3 = **0.75** kN/m^2 × **3** m = **2.25** kN/m

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	**14.6** kN	**5.62** kN	**20.2** kN	**28.1** kN
Right reaction	**14.6** kN	**5.62** kN	**20.2** kN	**28.1** kN

Design shear force



Design shear force V_{Ed} = **28.1** kN

Design shear resistance $V_{c,Rd}$ = **321** kN

Utilisation V_{Ed} / $V_{c,Rd}$ = **9** % **OK**

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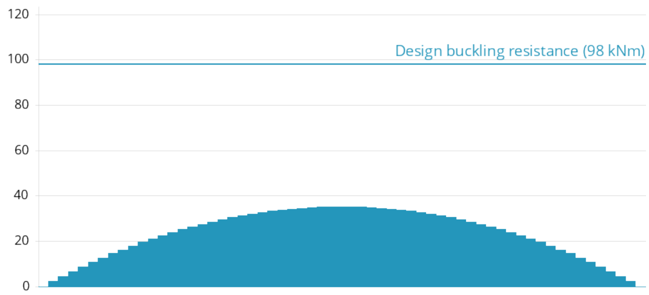
	Calcs for Connor McKitrich	Date 31 May 2023

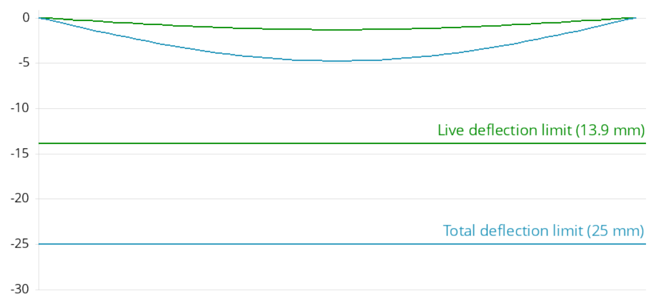
Design bending moment



Bending moments (kNm)

Design bending moment, major axis	M_{Ed} = **35.1** kNm
Design resistance for bending	$M_{c,Rd}$ = **156** kNm
Bending utilisation	M_{Ed} / $M_{c,Rd}$ = **23** % **OK**
Design resistance for buckling	$M_{b,Rd}$ = **98** kNm
Buckling utilisation	M_{Ed} / $M_{b,Rd}$ = **36** % **OK**

Deflection



Live load deflection (green) and total load deflection (blue) in mm

Variable action deflection limit	Δ_Q = **13.9** mm	
Variable action deflection	δ_Q = **1.3** mm	**OK**
Total deflection limit	Δ_{G+Q} = **25** mm	
Total deflection	δ_{G+Q} = **4.8** mm	**OK**

Section properties

Elastic modulus - major axis, yy	W_{el} = **504** cm^3
Plastic modulus - major axis, yy	W_{pl} = **566** cm^3
Second moment of area - major axis, yy	I_y = **6,540** cm^4
Second moment of area - minor axis, zz	I_z = **677** cm^4
Warping constant	I_w = **0.103** dm^6
Torsional constant	I_T = **23.9** cm^4
Area of section	A = **5,480** mm^2

Factors and design values of material coefficients (EN 1993-1-1:2005 and National Annex)

Young's modulus of elasticity	E = **210,000** N/mm^2	cl.3.2.6
Poisson's ratio in elastic stage	ν = **0.3**	cl.3.2.6
Shear modulus	G_s = **81,000** N/mm^2	cl.3.2.6
Partial factor for resistance of cross-sections	γ_{M0} = **1**	cl.6.1(1)B / BS-EN NA
Partial factor for resistance to instability	γ_{M1} = **1**	cl.6.1(1)B / BS-EN NA
Factor for shear area	η = **1**	EN 1993-1-5:2006 cl.5.1(2) / BS-EN NA
Limiting non dimensional slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.4**	cl.6.3.2.3(1) / BS-EN NA

Beta factor for buckling reduction factor calculation	β = **0.75**	cl.6.3.2.3(1) / BS-EN NA

Yield strength

Nominal yield strength for S275 grade and nominal section thickness 12.70 mm	f_y = **275** N/mm²	Tata blue book

Section classification (EN 1993-1-1:2005 cl.5.5)

Epsilon	ε = **0.924**	EN 1993-1-1:2005 Table 5.2
Flange ratio for local buckling	c_f / t_f = **4.92**	
Flange ratio limit for class 1	$9\,\varepsilon$ = **8.32**	Table 5.2 (sheet 2 of 3)
Flange class	$Class_f$ = **1**	
Web ratio for local buckling	c_w / t_w = **30.4**	
Web ratio limit for class 1	$72\,\varepsilon$ = **66.6**	Table 5.2 (sheet 1 of 3)
Web class	$Class_w$ = **1**	
Section class	Class = **1**	

Shear resistance (EN 1993-1-1:2005 cl.6.2.6)

Height of web	h_w = **234** mm	
Shear area for I and H sections	A_v = **2,020** mm²	cl.6.2.6 (3)
Design shear resistance	$V_{pl,Rd}$ = **321** kN	eq (6.18)

Shear buckling (EN 1993-1-5:2006 cl.5)

The shear buckling resistance for webs should be verified according to Section 5 of EN 1993-1-5 if $(h_w / t_w) > (72\,\varepsilon / \eta)$

Web ratio for shear buckling	h_w / t_w = **32.5**	EN 1993-1-5:2006 cl.5.1 (2)
Shear buckling limit	$72\,\varepsilon / \eta$ = **66.6**	EN 1993-1-5:2006 cl.5.1 (2)

$(h_w / t_w) \le (72\,\varepsilon / \eta)$ therefore shear buckling calculation not required

Bending resistance (EN 1993-1-1:2005 cl.6.2.5)

The shear force (28 kN) is less than half of the plastic shear resistance (321 kN / 2 = 161 kN), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus	W_{pl} = **566,000** mm³	
Design bending resistance	$M_{c,Rd}$ = **156** kNm	eq (6.13)

Design buckling resistance (EN 1993-1-1:2005 cl.6.3.2)

C1 factor	C1 = **1**	
Shear modulus of elasticity	G_s = **81,000** N/mm²	cl.3.2.6 (1)
Buckling length	L_{cr} = **5,000** mm	
Critical buckling moment	M_{CR} = **125** kNm	NCCI SN003b-EN-EU
Class 1 section, therefore use plastic modulus	W_{pl} = **566,000** mm³	cl.6.3.2.1(3)
Non-dimensional slenderness ratio	$\bar{\lambda}_{LT}$ = **1.12**	cl.6.3.2.2 (1)
Depth to width ratio for buckling curve	h / b = **1.76**	
Buckling curve for h / b ratio	Buckling curve = **b**	Table 6.5 / BS-EN NA
Imperfection factor for buckling curve b	α_{LT} = **0.34**	Table 6.3 / BS-EN NA

Intermediate factor for reduction factor calculation	Φ_{LT} = **1.09**	cl.6.3.2.3 (1)
Buckling reduction factor	χ_{LT} = **0.629**	eq (6.57)
Correction factor for moment distribution	k_c = **1**	Table 6.6
Moment distribution modification factor	f = **1**	cl.6.3.2.3 (2)
Modified buckling reduction factor	$\chi_{LT,mod}$ = **0.629**	eq (6.58)
Design buckling resistance	$M_{b,Rd}$ = **98** kNm	eq (6.55)

Notes

C1 value conservatively taken as 1.0

Ends of beam are to be laterally restrained. Ends of beams can be laterally restrained using one of the following methods;

1) End of beam built into masonry wall.
2) End of beam fixed to a masonry wall.
3) End of beam fixed to a column or a beam.

The designer is to ensure that the proposed detail adequately ensures that the end of the beam is laterally restrained.

No allowance has been made for destabilising loads which are outside the scope of these calculations (Destabilising loads would not normally occur in a traditional masonry structure)

STEEL MASONRY SUPPORT

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

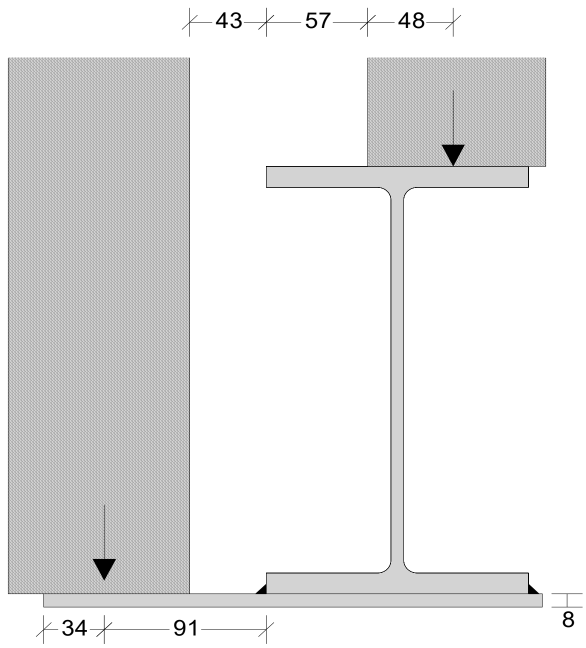
Tedds calculation version 1.0.03

Design summary

Overall design status PASS

Overall design utilisation 0.771

Description	Unit	Allowable	Applied	Utilisation	Result
Heel moment	kNm/m	2.933	0.150	0.051	PASS
Deflection	mm	1.8	0.1	0.029	PASS
Weld capacity	kN/m	945.2	240.6	0.255	PASS
Shear force (major axis)	kN	310.4	45.8	0.148	PASS
Bending (major-axis)	kNm	95.0	57.3	0.603	PASS
Bending (minor axis)	kNm	38.8	1.9	0.048	PASS
Warping	kNm	18.9	1.1	0.056	PASS
Bending and torsion				0.735	PASS
Plastic interaction				0.239	PASS
Torsion beam rotation	deg	2.00	1.39	0.695	PASS
Torsion beam deflection	mm	10.0	7.7	0.771	PASS



Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resist. of members to instability γ_{M1} = **1**

Resistance of joints γ_{M2} = **1.25**

Partial factor for permanent action γ_G = **1.35**

Partial factor for variable action γ_Q = **1.50**

Partial factor for permanent action (favourable) γ_{G_fav} = **1.00**

Partial factor for variable action (favourable) γ_{Q_fav} = **0.00**

Steel beam section details

Torsion beam section type — UB 254x146x43

Nominal yield strength — $f_y = f_{y,tb} = \mathbf{275}$ N/mm^2

Nominal ultimate tensile strength — $f_u = f_{u,tb} = \mathbf{410}$ N/mm^2

Masonry support section details

Section type — Plate 280x8(125)

Steel grade - EN 10025-2:2004 — S275

Nominal thickness — $t_{nom,sb} = t_{plate} = \mathbf{8}$ mm

Nominal yield strength — $f_{y,sb} = \mathbf{275}$ N/mm^2

Nominal ultimate tensile strength — $f_{u,sb} = \mathbf{410}$ N/mm^2

Modulus of elasticity — $E_{sb} = \mathbf{210000}$ N/mm^2

Total length of plate — $l_{plate} = \mathbf{280}$ mm

Length of plate beyond outer edge of torsion beam — $l_h = \mathbf{125}$ mm

Supported materials detail

Density of masonry on torsion beam — $\rho_{m,tb} = \mathbf{20.0}$ kN/m^3

Width of masonry on torsion beam — $b_{m,tb} = \mathbf{100}$ mm

Height of masonry on torsion beam — $h_{m,tb} = \mathbf{600}$ mm

Eccentricity of torsion beam masonry — $e_{load,tb} = \mathbf{105}$ mm

Eccentricity of torsion beam material — $e_{tb} = \mathbf{57}$ mm

Add perm. force torsion beam (not masonry) — $G_{k,add,tb} = \mathbf{5.0}$ kN/m

Add var. force torsion beam (not masonry) — $Q_{k,add,tb} = \mathbf{5.0}$ kN/m

Density of masonry on support beam — $\rho_{m,sb} = \mathbf{20.0}$ kN/m^3

Width of masonry on support beam — $b_{m,sb} = \mathbf{102}$ mm

Height of masonry on support beam — $h_{m,sb} = \mathbf{600}$ mm

Eccentricity of support beam masonry — $e_{load,sb} = \mathbf{91}$ mm

Geometry

Cavity width — $b_{cavity} = \mathbf{100}$ mm

Supported width of masonry — $d_m = l_h + t_{shim} + e_{tb} - b_{cavity} = \mathbf{82}$ mm

Biaxial stress effects in the plate (SCI-P-110)

Maximum overall bending moment — $M_{y,Ed} = \mathbf{57.3}$ kNm

Dist to NA combined section (CoG torsion beam) — $z_{na,all} = (h_{tb} + t_{plate}) \times A_{pl} / (2 \times (A_{tb} + A_{pl})) = \mathbf{39}$ mm

Second moment of area of combined section — $I_{y,all} = (I_{ytb} + A_{tb} \times z_{na,all}^2)+ A_{pl} \times (h_{tb} / 2 + t_{plate} / 2 - z_{na,all})^2 = \mathbf{9390}$ cm^4

Elastic section modulus of combined section — $Z_{y,all} = I_{y,all} / (h_{tb} / 2 + t_{plate} - z_{na,all}) = \mathbf{948.82}$ cm^3

Section modulus of plate — $Z_{y,plate} = 1m \times t_{plate}^2 / (6 \times 1m) = \mathbf{10.67}$ cm^3/m

Force of masonry on support plate — $F_1 = (b_{m,sb} \times h_{m,sb} \times \rho_{m,sb} + G_{k,add,sb}) \times \gamma_G + Q_{k,add,sb} \times \gamma_Q = \mathbf{1.7}$ kN/m

Bending at heel — $M_{y,Ed,plate} = F_1 \times e_{load,sb} = \mathbf{0.2}$ kNm/m

Moment capacity of plate — $M_{y,Rd,plate} = Z_{y,plate} \times f_{y,sb} / \gamma_{M0} = \mathbf{2.9}$ kNm/m

PASS - Moment capacity of plate exceeds applied moment

Longitudinal stress due to overall bending — $\sigma_1 = M_{y,Ed} / Z_{y,all} = \mathbf{60.4}$ N/mm^2

Constant relating to Von Mises curve — $c_{fp} = (4 \times f_{y,sb}^2 - 3 \times \sigma_1^2)^{0.5} = \mathbf{540.0}$ N/mm^2

Transverse bending stress ratio limit — $\alpha_{ts} = (c_{fp}^2 - \sigma_1^2) / (2 \times c_{fp} \times f_{y,sb}) = \mathbf{0.970}$

Transverse bending stress ratio — $\alpha_{ls} = M_{y,Ed,plate} / M_{y,Rd,plate} = \mathbf{0.051}$

PASS - Transverse bending stress ratio less than allowable limit

Deflection of plate

Unfactored force on support angle — $F_{1ser} = (b_{m,sb} \times h_{m,sb} \times \rho_{m,sb} + G_{k,add,sb}) + Q_{k,add,sb} = \mathbf{1.2}$ kN/m

Distance from weld to load position — $a_m = e_{load,sb} = \mathbf{91}$ mm

Length of load resultant to edge of plate — $b_m = l_h - e_{load,sb} = \mathbf{34}$ mm

Dist from weld to load position as ratio of length — $a_l = a_m / (a_m + b_m) = \mathbf{0.728}$

Effective second moment of area — $I_{eff\_def} = t_{plate}^3 / 12 = \mathbf{42667}$ mm^4/m

Deflection at end of plate — $\delta = (a_l^2 \times (3 - a_l) / 6) \times (F_{1ser} \times (a_m + b_m)^3) / (E_{sb} \times I_{eff\_def}) = \mathbf{0.05}$ mm

Deflection limit — $\delta_{lim} = \min((1 + d_m / b_{cavity}) \times 1\text{mm}, 2\text{mm}) = \mathbf{1.82}$ mm

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Shear force at weld position — $F_A = F_1 \times \max((1 + (3 \times e_{load,sb}) / (2 \times b_{tb} / 2)), 1.4) = \mathbf{4.7}$ kN/m

Maximum possible force in plate — $F_p = (l_h + \min(b_{tb}, l_{plate} - l_h)) \times t_{plate} \times f_{y,sb} = \mathbf{599.1}$ kN

Longitudinal shear between beam and plate — $F_l = 2 \times F_p / L = \mathbf{239.6}$ kN/m

Horizontal shear between beam and plate — $F_h = F_1 \times e_{load,sb} / (s_{weld} / 2 + t_{plate} / 2) = \mathbf{21.5}$ kN/m

Resultant weld force — $F_{w,Ed} = (F_A^2 + F_l^2 + F_h^2)^{0.5} = \mathbf{240.6}$ kN/m

Leg length of weld — $s_{weld} = \mathbf{6.00}$ mm

Throat thickness of weld — $a_{weld} = 1 / \sqrt{(2)} \times s_{weld} = \mathbf{4.24}$ mm

Length of weld per metre run — $l_{weld} = \mathbf{1000}$ mm/m

Ultimate tensile strength used for weld — $f_{u,weld} = \min(f_{u,sb}, f_{u,tb}) = \mathbf{410.0}$ N/mm^2

Correlation factor (table 4.1) — $\beta_w = \mathbf{0.85}$

Design shear strength — $f_{vw,d} = f_{u,weld} / (\sqrt{(3)} \times \beta_w \times \gamma_{M2}) = \mathbf{222.8}$ N/mm^2

Design resistance of weld — $F_{w,Rd} = f_{vw,d} \times a_{weld} = \mathbf{945.2}$ kN/m

PASS - weld capacity exceeds applied force

Eccentricities

Distance to shear centre of torsion beam — $e_{0,tb} = \mathbf{0}$ mm

Eccentricity of support beam masonry — $e_{m,sb} = e_{load,sb} + b_{tb} / 2 = \mathbf{165}$ mm

Eccentricity of torsion beam masonry — $e_{m,tb} = b_{tb} / 2 - e_{load,tb} = \mathbf{-31}$ mm

Eccentricity of support beam — $e_{b,sb} = c_{zsb} + b_{tb} / 2 = \mathbf{59}$ mm

Eccentricity of torsion beam — $e_{b,tb} = \mathbf{0}$ mm

Torsional loading ULS (unfavourable)

Loading of support beam masonry — $w_{sb} = (h_{m,sb} \times b_{m,sb} \times \rho_{m,sb}) \times \gamma_G = \mathbf{1.65}$ kN/m

Loading of torsion beam masonry — $w_{tb} = (h_{m,tb} \times b_{m,tb} \times \rho_{m,tb} + G_{k,add,tb}) \times \gamma_G + Q_{k,add,tb} \times \gamma_Q = \mathbf{15.87}$ kN/m

Self weight of support beam — $w_{sw,sb} = A_{pl} \times \rho_{SEC3} \times g_{acc} \times \gamma_G = \mathbf{0.23}$ kN/m

Self weight of torsion beam — $w_{sw,tb} = A_{tb} \times \rho_{SEC3} \times g_{acc} \times \gamma_G = \mathbf{0.57}$ kN/m

Torsional loading ULS (favourable)

Loading of support beam masonry — $w_{sb\_fav} = (h_{m,sb} \times b_{m,sb} \times \rho_{m,sb}) \times \gamma_{G\_fav} = \mathbf{1.22}$ kN/m

Loading of torsion beam masonry — $w_{tb\_fav} = (h_{m,tb} \times b_{m,tb} \times \rho_{m,tb} + G_{k,add,tb}) \times \gamma_{G\_fav} + Q_{k,add,tb} \times \gamma_{Q\_fav} = \mathbf{6.20}$ kN/m

Self weight of support beam — $w_{sw,sb\_fav} = A_{pl} \times \rho_{SEC3} \times g_{acc} \times \gamma_{G\_fav} = \mathbf{0.17}$ kN/m

Self weight of torsion beam — $w_{sw,tb\_fav} = A_{tb} \times \rho_{SEC3} \times g_{acc} \times \gamma_{G\_fav} = \mathbf{0.42}$ kN/m

Torsional loading SLS (unfavourable)

Loading of support beam masonry — $w_{sb,ser} = h_{m,sb} \times b_{m,sb} \times \rho_{m,sb} = \mathbf{1.22}$ kN/m

Loading of torsion beam masonry — $w_{tb,ser} = h_{m,tb} \times b_{m,tb} \times \rho_{m,tb} + G_{k,add,tb} + Q_{k,add,tb} = \mathbf{11.20}$ kN/m

Self weight of support beam — $w_{sw,sb,ser} = A_{pl} \times \rho_{SEC3} \times g_{acc} = \mathbf{0.17}$ kN/m

Self weight of torsion beam — $w_{sw,tb,ser} = A_{tb} \times \rho_{SEC3} \times g_{acc} = \mathbf{0.42}$ kN/m

Torsional effects

Applied torque (ULS +ve ecc. unfav, -ve ecc. fav) $T_{d,w,fav1}$ = abs($w_{sb} \times e_{m,sb} + w_{tb\_fav} \times e_{m,tb} + w_{sw,sb} \times e_{b,sb} + w_{sw,tb} \times e_{b,tb}$) = **0.09** kNm/m

Applied torque (ULS +ve ecc. fav, -ve ecc. unfav) $T_{d,w,fav2}$ = abs($w_{sb\_fav} \times e_{m,sb} + w_{tb} \times e_{m,tb} + w_{sw,sb\_fav} \times e_{b,sb} + w_{sw,tb\_fav} \times e_{b,tb}$) = **0.29** kNm/m

Applied torque (ULS all unfavourable) $T_{d,w}$ = abs($w_{sb} \times e_{m,sb} + w_{tb} \times e_{m,tb} + w_{sw,sb} \times e_{b,sb} + w_{sw,tb} \times e_{b,tb}$) = **0.21** kNm/m

Total torque (ULS) T_d = max($T_{d,w}$, $T_{d,w,fav1}$, $T_{d,w,fav2}$) × L = **1.43** kNm

Applied torque (SLS) $T_{d,w,ser}$ = abs($w_{sb,ser} \times e_{m,sb} + w_{tb,ser} \times e_{m,tb} + w_{sw,sb,ser} \times e_{b,sb} + w_{sw,tb,ser} \times e_{b,tb}$) = **0.14** kNm/m

Total torque (SLS) $T_{d,ser} = T_{d,w,ser} \times L$ = **0.70** kNm

STEEL BEAM TORSION DESIGN (EN1993)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 1.0.05

Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Section details

Section type UB 254x146x43 (BS4-1)

Steel grade - EN 10025-2:2004 S275

Nominal thickness of element t_{nom} = max(t_f, t_w) = **12.7** mm

Nominal yield strength f_y = **275** N/mm^2

Nominal ultimate tensile strength f_u = **410** N/mm^2

Modulus of elasticity E = **210000** N/mm^2

Shear centre

Distance between flange shear centres $h_s = h - t_f$ = **246.9** mm

Shear centre (above bottom flange centroid) $e_{s,bf} = h_s / 2$ = **123.5** mm

Analysis results

Design bending moment - major axis $M_{y,Ed}$ = **57.3** kNm

Design shear force - major axis $V_{y,Ed}$ = **45.8** kN

Classification

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section c = d = **219** mm

$c / t_w = 30.4 = 32.9 \times \varepsilon <= 72 \times \varepsilon$ Class 1

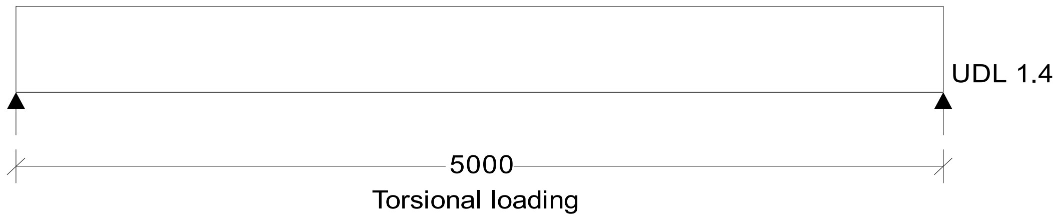
Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **62.5** mm

$c / t_f = 4.9 = 5.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Torsional loads

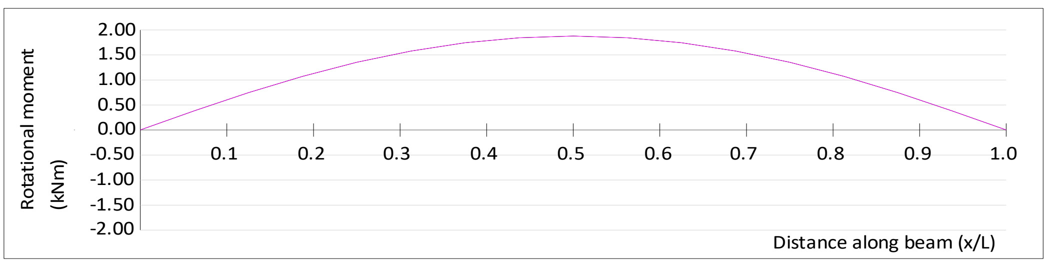


Load No.	Load type	Load (kNm)	Distance along beam (mm)
1	UDL	1.4	-

Rotation ϕ (SCI P385 Appendix C Case 4)

$\phi = T_d \times a^2 / (G_{SEC3} \times I_t \times L) \times ((x \times L - x^2) / (2 \times a^2) + \cosh(x / a) - \tanh(L / (2 \times a)) \times \sinh(x / a) - 1)$

Additional minor moment due to rotation, $M_{z,add,Ed} = M_{y,Ed} \times \phi$

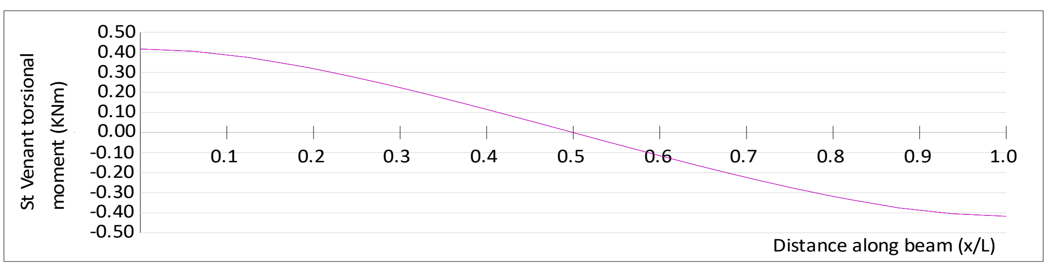


Additional minor design moment due to rotation $M_{z,add,Ed}$ = **1.88** kNm

St Venant ϕ' (SCI P385 Appendix C Case 4)

$\phi' = T_d \times a / (G_{SEC3} \times I_t \times L) \times (L / (2 \times a) - x / a + \sinh(x / a) - \tanh(L / (2 \times a)) \times \cosh(x / a))$

Design value of the internal St Venant torsion moment $T_{t,Ed} = G_{SEC3} \times I_t \times \phi'$

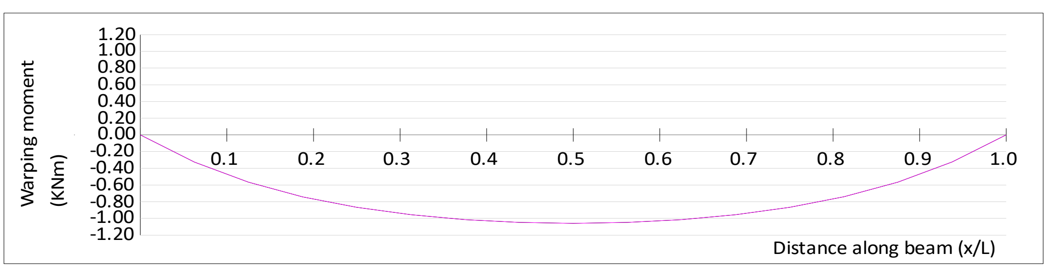


St Venant torsion design moment $T_{t,Ed}$ = **0.42** kNm

Warping ϕ'' (SCI P385 Appendix C Case 4)

$\phi'' = T_d / (G_{SEC3} \times I_t \times L) \times (-1 + \cosh(x / a) - \tanh(L / (2 \times a)) \times \sinh(x / a))$

Warping design moment, $M_{w,Ed} = E_{SEC3} \times I_w \times \phi'' / h_s$

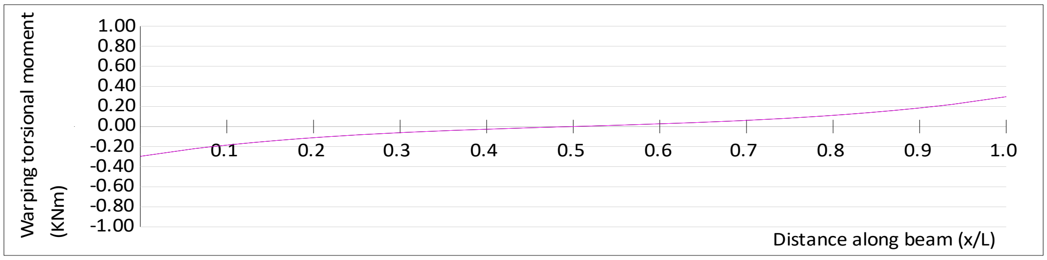


Warping design moment $M_{w,Ed}$ = **1.06** kNm

Warping torsional ϕ''' (SCI P385 Appendix C Case 4)

$\phi''' = T_d / (G_{SEC3} \times I_t \times L \times a) \times (\sinh(x / a) - \tanh[L / (2 \times a)] \times \cosh(x / a))$

Warping torsional design moment, $T_{w,Ed} = E_{SEC3} \times I_w \times \phi'''$



Warping torsional design moment $T_{w,Ed}$ = **0.30** kNm

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **234.2** mm

η = **1.000**

$h_w / t_w = 32.5 = 35.2 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **45.81** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **2020** mm^2

Design shear resistance - cl 6.2.6(2) $V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **320.8** kN

Shear stress due to St Venant torsion $\tau_{t,Ed} = T_{t,Ed} \times t_w / I_t$ = **12.57** N/mm^2

Reduced shear resistance due to torsion - eq 6.26 $V_{c,y,Rd} = V_{pl,T,y,Rd} = \sqrt{(1 - \tau_{t,Ed} / (1.25 \times (f_y / \sqrt{(3)}) / \gamma_{M0}))} \times V_{pl,y,Rd}$ = **310.4** kN

$V_{y,Ed} / V_{pl,T,y,Rd}$ = **0.148**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **57.3** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **155.7** kNm

$M_{y,Ed} / M_{pl,y,Rd}$ = **0.368**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Loading factor C_1 C_1 = **1.127**

Loading factor C_2 C_2 = **0.454**

Loading factor C_3 C_3 = **0.52**

Poissons ratio ν = **0.3**

Shear modulus $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained effective length L = **5000** mm

Distance from shear centre to level of load $z_g = (h_{tb} / 2 \times w_{tb} - h_{tb} / 2 \times w_{sb}) / (w_{tb} + w_{sb})$ = **105.3** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (k_{z,cr} \times L^2) \times (\sqrt{((k_{z,cr} / k_{w,cr})^2 \times I_w / I_z + L^2 \times G \times I_t /}$
$(\pi^2 \times E \times I_z) + (C_2 \times z_g)^2) - (C_2 \times z_g))$ = **113.9** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.169**

Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.4**

Lateral torsional buckling cannot be ignored

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5 — b

Imperfection factor - Table 6.3 — α_{LT} = **0.34**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **1.144**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.597**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.979**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.610**

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **95** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.603**

PASS - Design buckling resistance exceeds design buckling moment

Check bending moment - Section 6.2.5

Design bending moment — $M_{z,Ed,total} = M_{z,Ed} + M_{z,add,Ed}$ = **1.9** kNm

Design bending resistance moment - eq 6.13 — $M_{c,z,Rd} = M_{pl,z,Rd} = W_{pl.z} \times f_y / \gamma_{M0}$ = **38.8** kNm

$M_{z,Ed,total} / M_{pl,z,Rd}$ = **0.048**

PASS - Design bending resistance moment exceeds design bending moment

Check warping moment

Warping moment in flange — $M_{w,Ed}$ = **1.06** kNm

Plastic modulus of flange — $W_{pl.f} = t_f \times b^2 / 4$ = **68.89** cm^3

Design warping resistance of flange — $M_{w,Rd} = W_{pl.f} \times f_y / \gamma_{M0}$ = **18.94** kNm

$M_{w,Ed} / M_{w,Rd}$ = **0.056**

PASS - Bending resistance in one flange exceeds the design warping moment

Combined bending and torsion (EN1993-6 Annex A)

Equiv. uniform moment factor (parabolic curve) — C_{mz} = **0.95**

Characteristic moment resistance - y axis — $M_{y,Rk} = W_{pl.y} \times f_y$ = **155.7** kNm

Characteristic moment resistance - z axis — $M_{z,Rk} = W_{pl.z} \times f_y$ = **38.8** kNm

Characteristic warping resistance — $M_{w,Rk} = W_{pl.f} \times f_y$ = **18.9** kNm

Interaction factors — $k_w = 0.7 - 0.2 \times M_{w,Ed} / (M_{w,Rk} / \gamma_{M1})$ = **0.69**

$k_{zw} = 1 - M_{z,Ed,total} / (M_{z,Rk} / \gamma_{M1})$ = **0.95**

$k_\alpha = 1 / (1 - M_{y,Ed} / M_{cr})$ = **2.01**

Interaction formula eqn A.1 — $M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + C_{mz} \times M_{z,Ed,total} / (M_{z,Rk} / \gamma_{M1}) + k_w \times k_{zw} \times k_\alpha \times M_{w,Ed} / (M_{w,Rk} / \gamma_{M1})$ = **0.735**

PASS - Combined bending and torsion check satisfied

Plastic verification - Exp.6.41

$(M_{y,Ed} / M_{pl,y,Rd})^2 + M_{z,Ed,total} / M_{pl,z,Rd} + M_{w,Ed} / M_{w,Rd}$ = **0.239**

PASS - Plastic interaction criterion is less than 1.0

Serviceability limit checks

Rotation limit — $\phi_{ser,lim}$ = **2.00** deg

Rotation of torsion beam — $\phi_{ser} = M_{z,add,Ed} \times 180 / (M_{y,Ed} \times \gamma_G \times \pi)$ = **1.39** deg

PASS - Rotation limit exceeds rotation in torsion beam

Vertical deflection limit — $\delta_{v,lim}$ = **10.0** mm

SLS loading on beam — $f_{d,ser} = w_{sb,ser} + w_{tb,ser} + w_{sw,sb,ser} + w_{sw,tb,ser}$ = **13.02** kN/m

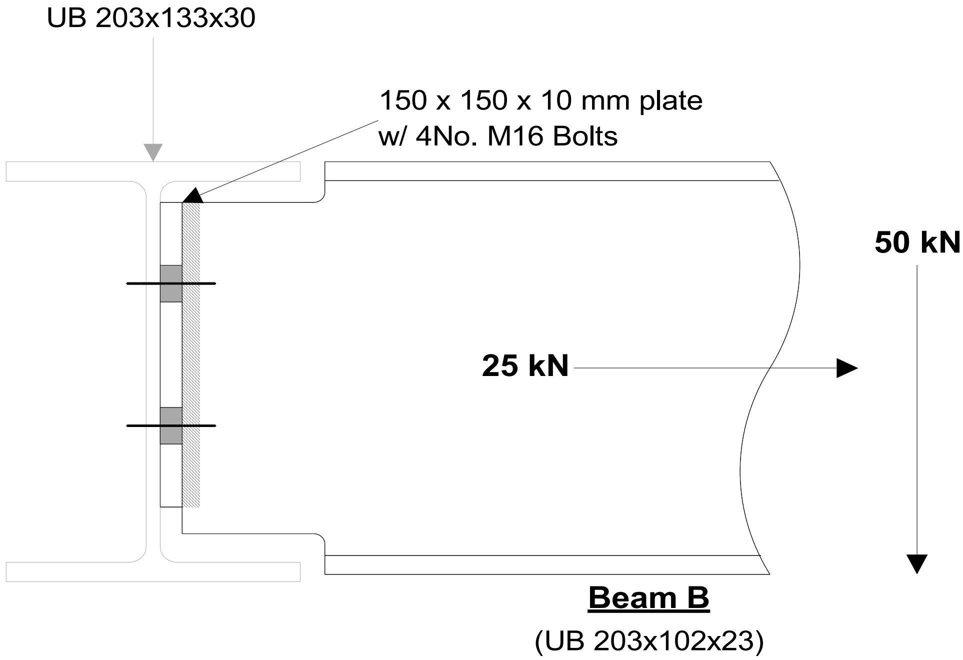
Vertical deflection of torsion beam — $\delta_v = 5 \times f_{d,ser} \times L^4 / (384 \times E_{sb} \times I_{ytb})$ = **7.7** mm

PASS - Vertical deflection limit exceeds vertical deflection in torsion beam

STEEL CONNECTION DESIGN

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009, and EN1993-1-8:2005 incorporating Corrigenda December 2005, September 2006 and July 2009, and the UK National Annex.

Tedds calculation version 1.2.01



Connection details

Connection type **Partial depth end plate**

Number of supported beams **1 supported beam**

Partial factors

Resistance of cross-section γ_{M0} = **1.00**

Resistance of members to instability γ_{M1} = **1.00**

Cross-sections in tension to fracture $\gamma_{M2,c}$ = **1.10**

Resistance of bolts $\gamma_{M2,b}$ = **1.25**

Structural integrity $\gamma_{M,u}$ = **1.10**

Supporting beam details

Section name **UB 203x133x30**

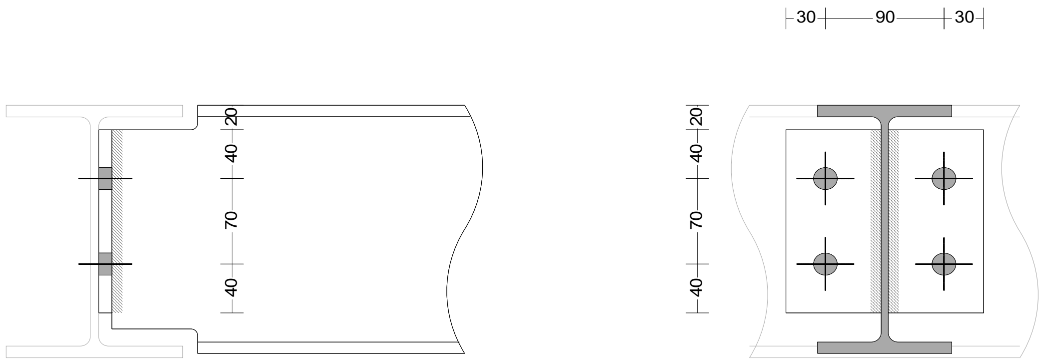
Steel grade **S275**

Yield strength f_y = **275** N/mm^2

Ultimate strength f_u =**410** N/mm^2

PlanningApplications.com Summer House, Upper Court Road Woldingham SURREY CR3 7BF support@planningapplications.com 07922 148 701	Project Connection 1				Job no. 739	
	Calcs for Connor McKitrich				Start page no./Revision 2	
	Calcs by SB	Calcs date 30/05/2023	Checked by DB	Checked date 30/05/2023	Approved by SB	Approved date 30/05/2023

Supported Beam B



Summary Results

Check	Description	Units	Design Force	Design Resistance	Utilisation	
1	Recommended detailing practices					PASS
2	Supported beam - Welds	kN	28	212.7	0.131	PASS
4	Supported beam - Web in shear	kN	50	115.7	0.432	PASS
5	Supported beam - Resistance at notch	kNm	3.8	6.6	0.569	PASS
6	Supported beam - Local stability notch					PASS
8	Connection - Bolt group	kN	50	192.9	0.259	PASS
9	Connection - End plate in shear	kN	50	375.1	0.133	PASS
10	Supporting beam - Shear	kN	25	177.8	0.141	PASS
11	Tying resistance - Plate and bolts	kN	25	183.5	0.136	PASS
12	Tying resistance - Supported beam web	kN	25	301.9	0.083	PASS

Design forces

Design shear — V_{Ed1} = **50** kN

Design tying force — F_{Ed1} = **25** kN

Supported beam details

Section name — **UB 203x102x23**

Steel grade — **S275**

Yield strength — $f_{y,b}$ = **275** N/mm^2

Ultimate strength — $f_{u,b}$ = **410** N/mm^2

Correlation factor — $\beta_{w,b}$ = **0.85**

End plate details

Plate height — h_p = **150** mm

Plate width — b_p = **150** mm

Plate thickness — t_p = **10** mm

Plate grade — **S275**

Yield strength — $f_{y,p}$ = **275** N/mm^2

Ultimate strength $f_{u,p}$ = **410** N/mm^2

Correlation factor $\beta_{w,p}$ = **0.85**

Bolt details

Number of bolt rows $n_{1,1}$ = **2**

Total number of bolts n_b = **4**

End distance e_1 = **40** mm

Edge distance e_2 = **30** mm

Pitch p_1 = **70** mm

Gauge p_3 = **90** mm

Bolt hole d_0 = **18** mm

Bolt size **M16**

Bolt grade **8.8**

Yield strength $f_{y,bolt}$ = **640** N/mm^2

Ultimate strength $f_{u,bolt}$ = **800** N/mm^2

Check 1: Recommended detailing practice

Minimum plate height $0.6 \times h_b$ = **121.9** mm

Actual plate height h_p = **150** mm

Maximum depth to plate **50 mm**

Actual depth to plate d_p = **20** mm

Maximum plate thickness **10 mm**

Actual plate thickness t_p = **10** mm

Minimum bolt gauge **90 mm**

Actual bolt gauge p_3 = **90** mm

Top Notch

Depth of notch d_{nt} = **20** mm

Length of notch l_n = **65** mm

Minimum vertical clearance $Max(t_{f,b} + r_b, t_f + r)$ = **17.2** mm

Actual vertical clearance d_{nt} = **20** mm

Minimum horizontal clearance **10 mm**

Actual horizontal clearance $l_n - (b - t_w) / 2 + t_p$ = **11.2** mm

Bottom Notch

Depth of notch d_{nb} = **20** mm

Length of notch l_n = **65** mm

Minimum vertical clearance $t_{f,b,A\_c1} + r_b$ = **16.9** mm

Actual vertical clearance d_{nb} = **20** mm

PASS - Recommended detailing practices are met

Check 2: Supported beam - Welds

Weld leg size s_w = **8.0** mm

Minimum weld throat thickness $0.4 \times t_{w,b}$ = **2.2** mm

Effective weld throat thickness $a_w = 0.7 \times s_w$ = **5.6** mm

Correlation factor $\beta_w = Min(\beta_{w,b}, \beta_{w,p})$ = **0.85**

Design shear strength $f_{vw,d} = Min(f_{u,b},f_{u,p}) / \sqrt{(3)} / (\beta_w \times \gamma_{M2,c})$ = **253.17** N/mm^2

Design resistance $F_{w,Rd} = f_{vw,d} \times a_w \times h_p$ = **212.66** kN

Design weld force $F_{w,Ed} = \sqrt{(V_{Ed1}^2 + F_{Ed1}^2)} / 2$ = **27.95** kN

Utilisation $F_{w,Ed} / F_{w,Rd}$ = **0.131**

PASS - Weld throat thickness greater than required

Check 4: Supported beam - Web in shear

Shear area — $A_v = 0.9 \times h_p \times t_{w,b} = \mathbf{729}$ mm^2

Plastic shear resistance of beam web — $V_{pl,Rd} = A_v \times (f_{y,b} / \sqrt{(3)}) / \gamma_{M0} = \mathbf{115.74}$ kN

Design shear resistance — $V_{c,Rd} = V_{pl,Rd} = \mathbf{115.74}$ kN

Utilisation — $V_{Ed1} / V_{c,Rd} = \mathbf{0.432}$

PASS - Web shear resistance greater than design shear

Check 5: Supported beam - Resistance at notch

Double Notch (low shear, $V_{Ed} <= 0.5V_{pl,DN,Rd}$)

Shear area at notch — $A_{v,DN} = 0.9 \times (h_b - d_{nt} - d_{nb}) \times t_{w,b} = \mathbf{793}$ mm^2

Shear resistance at notch — $V_{pl,DN,Rd} = (A_{v,DN} \times f_{y,b}) / (\sqrt{(3)} \times \gamma_{M0}) = \mathbf{125.93}$ kN

Moment resistance at notch — $M_{v,DN,Rd} = f_{y,b} \times t_{w,b} / (6 \times \gamma_{M0}) \times (h_b - d_{nt} - d_{nb})^2 = \mathbf{6.59}$ kNm

Design moment at notch — $M_{v,Ed} = V_{Ed1} \times (t_p + l_n) = \mathbf{3.75}$ kNm

Utlilisation — $M_{v,Ed} / M_{v,N,Rd} = 0.569$

PASS - Notch resistance is greater than design force

Check 6: Supported beam - Local stability of notched beam

Double notch

Maximum notch depth — $h_b / 5 = \mathbf{40.6}$ mm

Actual notch depth — $Max(d_{nt}, d_{nb}) = \mathbf{20}$ mm

Maximum notch length ($h_b/t_{w,b} <= 54.3$) — $h_b = \mathbf{203.2}$ mm

Actual notch length — $l_n = \mathbf{65}$ mm

PASS - Local stability is accounted for

Check 8: Connection - Bolt group

Bolt tensile stress area — $A_s = \mathbf{157}$ mm^2

Bolt shear stress factor — $\alpha_v = \mathbf{0.6}$

Bolt shear resistance — $F_{v,Rd} = \alpha_v \times f_{u,bolt} \times A_s / \gamma_{M2,b} = \mathbf{60.29}$ kN

For the end plate — $\alpha_{b,p} = Min(e_1 / (3 \times d_0), p_1 / (3 \times d_0) - 1/4, f_{u,bolt} / f_{u,p}, 1) = \mathbf{0.74}$

$k_{1,p} = Min(2.8 \times e_2 / d_0 - 1.7, 1.4 \times p_3 / d_0 - 1.7, 2.5) = \mathbf{2.5}$

For the supporting member — $\alpha_{b,2} = Min(p_1 / (3 \times d_0) - 1/4, f_{u,bolt} / f_u, 1) = \mathbf{1}$

$k_{1,2} = Min(1.4 \times p_3 / d_0 - 1.7, 2.5) = \mathbf{2.5}$

Bearing on the end plate — $F_{b,Rd,p} = k_{1,p} \times \alpha_{b,p} \times f_{u,p} \times d_b \times t_p / \gamma_{M2,b} = \mathbf{97.19}$ kN

Bearing on the supporting member — $F_{b,Rd,2} = k_{1,2} \times \alpha_{b,2} \times f_u \times d_b \times t_w / \gamma_{M2,b} = \mathbf{83.97}$ kN

Minimum bearing resistance — $F_{b,Rd1} = Min(F_{b,Rd,p}, F_{b,Rd,2}) = \mathbf{83.97}$ kN

Resistance of the bolt group — $F_{Rd} = 0.8 \times n_b \times F_{v,Rd} = \mathbf{192.92}$ kN

Utilisation — $V_{Ed1} / F_{Rd} = \mathbf{0.259}$

PASS - Bolt group resistance is greater than design force

Check 9: Connection - End plate in shear

Net shear area — $A_{v,net} = t_p \times (h_p - n_{1,1} \times d_0) = \mathbf{1140}$ mm^2

Edge shear area — $A_{nt} = t_p \times (e_2 - d_0 / 2) = \mathbf{210}$ mm^2

Shear area from end bolt — $A_{nv} = t_p \times (h_p - e_1 - (n_{1,1} - 0.5) \times d_0) = \mathbf{830}$ mm^2

Gross section shear resistance — $V_{Rd,g} = (2 \times h_p \times t_p) / 1.27 \times f_{y,p} / (\sqrt{(3)} \times \gamma_{M0}) = \mathbf{375.05}$ kN

Net section shear resistance — $V_{Rd,n} = 2 \times A_{v,net,A\_c1} \times f_{u,plate,A\_c1} / (\sqrt{(3)} \times \gamma_{M2,c}) = \mathbf{490.64}$ kN

Block tearing resistance — $V_{Rd,b} = 2 \times (f_{u,p} \times A_{nt} / \gamma_{M2,c} + f_{y,p} \times A_{nv} / (\sqrt{(3)} \times \gamma_{M0})) = \mathbf{420.11}$ kN

End plate in-plane bending resistance — $h_p < 1.36p_3$ - No additional requirements

End plate shear resistance — $V_{Rd,pl,min} = Min(V_{Rd,g}, V_{Rd,n}, V_{Rd,b}) = \mathbf{375.05}$ kN

Utilisation _PlateShearUtilisationA_c1 = **0.133**

PASS - Shear resistance of end plate greater than design force

Check 10: Supporting beam - Shear

Distance from top bolt to flange $e_{1,t}$ = **60** mm

Distance from bottom bolt to flange $e_{1,b}$ = **77** mm

Minimum top distance $e_t = \mathrm{Min}(e_{1,t}, 5 \times d_b)$ = **60** mm

Minimum bottom distance $e_b = \mathrm{Min}(e_{1,b}, p_3 / 2, 5 \times d_b)$ = **45** mm

Shear area of supporting member $A_v = t_w \times (e_t + (n_{1,1} - 1) \times p_1 + e_b)$ = **1120** mm^2

Net shear area of supporting member $A_{v,net} = A_v - n_{1,1} \times d_0 \times t_w$ = **890** mm^2

Local shear resistance $V_{Rd,min} = \mathrm{Min}(A_v \times f_y / (\sqrt{(3)} \times \gamma_{M0}), A_{v,net} \times f_u / (\sqrt{(3)} \times \gamma_{M2,c}))$ = **177.82** kN

Utilisation $V_{Ed1}/2 / V_{Rd,min}$ = **0.141**

PASS - Beam shear resistance is greater than design force

Check 11: Tying resistance - Plate and bolts

Effective end distance $e_{1A} = \mathrm{Min}(e_1, 0.5 \times (p_3 - t_{w,b} - 2 \times a_w \times \sqrt{(2)}) + d_0/2)$ = **40** mm

Effective bolt pitch $p_{1A} = \mathrm{Min}(p_1, p_3 - t_{w,b} - 2 \times a_w \times \sqrt{(2)} + d_0)$ = **70** mm

Minimum end distance $e_{min} = e_2$ = **30** mm

Bolt factor k_2 = **0.9**

Distance from weld throat to bolt $m_w = (p_3 - t_{w,b} - 2 \times 0.8 \times a_w \times \sqrt{(2)}) / 2$ = **36** mm

$n_w = \mathrm{Min}(e_{min}, 1.25 \times m_w)$ = **30** mm

Width across bolt head points d_w = **26** mm

$e_w = d_w / 4$ = **6.5** mm

Effective length of equivalent T-stub $\Sigma l_{eff} = 2 \times e_{1A} + (n_{1,1} - 1) \times p_{1A}$ = **150.0** mm

Moment resistance of plate $M_{pl,1,Rd,u} = (0.25 \times \Sigma l_{eff} \times t_p^2 \times f_{u,p}) / \gamma_{M,u}$ = **1.4** kNm

$M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$ = **1.4** kNm

Mode 1 plate failure $F_{Rd,u,1} = (8 \times n_w - 2 \times e_w) \times M_{pl,1,Rd,u} / (2 \times m_w \times n_w - e_w \times (m_w + n_w))$ = **183.5** kN

Individual bolt resistance $F_{t,Rd,u} = k_2 \times f_{u,bolt} \times A_s / \gamma_{M,u}$ = **102.76** kN

Group bolt resistance $\Sigma F_{t,Rd,u} = n_b \times F_{t,Rd,u}$ = **411.05** kN

Mode 2 bolt and plate failure $F_{Rd,u,2} = (2 \times M_{pl,2,Rd,u} + n_w \times \Sigma F_{t,Rd,u}) / (m_w + n_w)$ = **229.32** kN

Mode 3 bolt failure $F_{Rd,u,3} = \Sigma F_{t,Rd,u}$ = **411.05** kN

Minimum resistance $F_{Rd,u,min} = \mathrm{Min}(F_{Rd,u,1}, F_{Rd,u,2}, F_{Rd,u,3})$ = **183.5** kN

Utilisation $F_{Ed1} / F_{Rd,u,min}$ = **0.136**

PASS - Tying resistance of plate and bolts is greater than design force

Check 12: Tying resistance - Supported beam web

Web resistance $F_{Rd,u} = (t_{w,b} \times h_p \times f_{u,b}) / \gamma_{M,u}$ = **301.91** kN

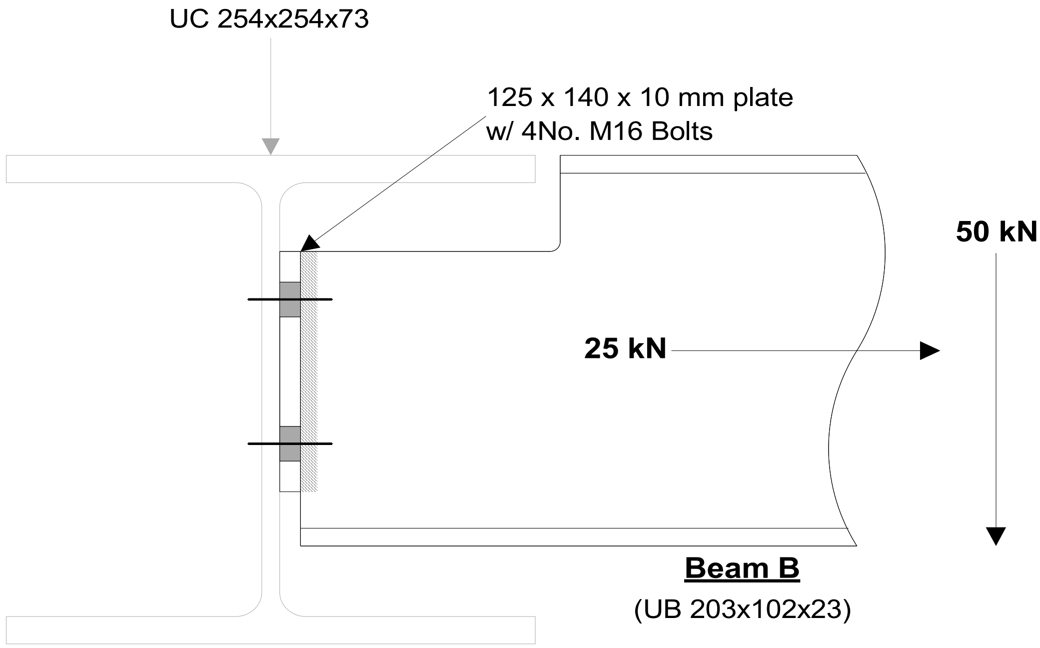
Utilisation $F_{Ed1} / F_{Rd,u}$ = **0.083**

PASS - Supported beam web tying resistance is greater than design force

STEEL CONNECTION DESIGN

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009, and EN1993-1-8:2005 incorporating Corrigenda December 2005, September 2006 and July 2009, and the UK National Annex.

Tedds calculation version 1.2.01



Connection details

Connection type	**Partial depth end plate**
Number of supported beams	**1 supported beam**

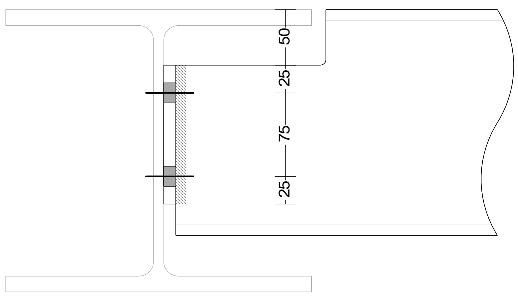
Partial factors

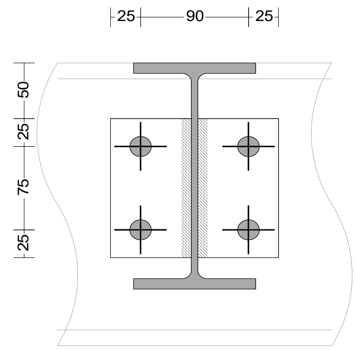
Resistance of cross-section	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Cross-sections in tension to fracture	$\gamma_{M2,c}$ = **1.10**
Resistance of bolts	$\gamma_{M2,b}$ = **1.25**
Structural integrity	$\gamma_{M,u}$ = **1.10**

Supporting beam details

Section name	**UC 254x254x73**
Steel grade	**S275**
Yield strength	f_y = **275** N/mm^2
Ultimate strength	f_u =**410** N/mm^2

Supported Beam B





Summary Results

Check	Description	Units	Design Force	Design Resistance	Utilisation	
1	Recommended detailing practices					PASS
2	Supported beam - Welds	kN	28	177.2	0.158	PASS
4	Supported beam - Web in shear	kN	50	96.5	0.518	PASS
5	Supported beam - Resistance at notch	kNm	6.8	9.3	0.727	PASS
6	Supported beam - Local stability notch					PASS
8	Connection - Bolt group	kN	50	192.9	0.259	PASS
9	Connection - End plate in shear	kN	50	312.5	0.160	PASS
10	Supporting beam - Shear	kN	25	266.3	0.094	PASS
11	Tying resistance - Plate and bolts	kN	25	155.4	0.161	PASS
12	Tying resistance - Supported beam web	kN	25	251.6	0.099	PASS

Design forces

Design shear V_{Ed1} = **50** kN

Design tying force F_{Ed1} = **25** kN

Supported beam details

Section name **UB 203x102x23**

Steel grade **S275**

Yield strength $f_{y,b}$ = **275** N/mm^2

Ultimate strength $f_{u,b}$ = **410** N/mm^2

Correlation factor $\beta_{w,b}$ = **0.85**

End plate details

Plate height h_p = **125** mm

Plate width b_p = **140** mm

Plate thickness t_p = **10** mm

Plate grade **S275**

Yield strength $f_{y,p}$ = **275** N/mm^2

Ultimate strength $f_{u,p}$ = **410** N/mm^2

Correlation factor $\beta_{w,p}$ = **0.85**

Bolt details

Number of bolt rows $n_{1,1}$ = **2**

Total number of bolts n_b = **4**

End distance e_1 = **25** mm

Edge distance e_2 = **25** mm

Pitch p_1 = **75** mm

Gauge p_3 = **90** mm

Bolt hole d_0 = **18** mm

Bolt size **M16**

Bolt grade **8.8**

Yield strength $f_{y,bolt}$ = **640** N/mm^2

Ultimate strength $f_{u,bolt}$ = **800** N/mm^2

Check 1: Recommended detailing practice

Minimum plate height $0.6 \times h_b$ = **121.9** mm

Actual plate height h_p = **125** mm

Maximum depth to plate **50 mm**

Actual depth to plate d_p = **50** mm

Maximum plate thickness **10 mm**

Actual plate thickness t_p = **10** mm

Minimum bolt gauge **90 mm**

Actual bolt gauge p_3 = **90** mm

Top Notch

Depth of notch d_{nt} = **50** mm

Length of notch l_n = **125** mm

Minimum vertical clearance $\text{Max}(t_{f,b} + r_b, t_f + r)$ = **26.9** mm

Actual vertical clearance d_{nt} = **50** mm

Minimum horizontal clearance **10 mm**

Actual horizontal clearance $l_n - (b - t_w) / 2 + t_p$ = **12** mm

PASS - Recommended detailing practices are met

Check 2: Supported beam - Welds

Weld leg size s_w = **8.0** mm

Minimum weld throat thickness $0.4 \times t_{w,b}$ = **2.2** mm

Effective weld throat thickness $a_w = 0.7 \times s_w$ = **5.6** mm

Correlation factor $\beta_w = \text{Min}(\beta_{w,b}, \beta_{w,p})$ = **0.85**

Design shear strength $f_{vw,d} = \text{Min}(f_{u,b},f_{u,p}) / \sqrt{(3)} / (\beta_w \times \gamma_{M2,c})$ = **253.17** N/mm^2

Design resistance $F_{w,Rd} = f_{vw,d} \times a_w \times h_p$ = **177.22** kN

Design weld force $F_{w,Ed} = \sqrt{(V_{Ed1}^2 + F_{Ed1}^2)} / 2$ = **27.95** kN

Utilisation $F_{w,Ed} / F_{w,Rd}$ = **0.158**

PASS - Weld throat thickness greater than required

Check 4: Supported beam - Web in shear

Shear area $A_v = 0.9 \times h_p \times t_{w,b}$ = **608** mm^2

Plastic shear resistance of beam web $V_{pl,Rd} = A_v \times (f_{y,b} / \sqrt{(3)}) / \gamma_{M0}$ = **96.45** kN

Design shear resistance $V_{c,Rd} = V_{pl,Rd}$ = **96.45** kN

Utilisation $V_{Ed1} / V_{c,Rd}$ = **0.518**

PASS - Web shear resistance greater than design shear

Check 5: Supported beam - Resistance at notch

Single Notch (low shear, V_{Ed} <= $0.5V_{pl,N,Rd}$)

Area of Tee section at notch A_{Tee} = **1724** mm^2

Elastic modulus of Tee section $W_{el,N,y}$ = **33782** mm^3

Shear area at notch $A_{v,N} = A_{Tee} - b_b \times t_{f,b} + (t_{w,b} + 2 \times r_b) \times t_{f,b} / 2$ = **873** mm^2

Shear resistance at notch $V_{pl,N,Rd} = (A_{v,N} \times f_{y,b}) / (\sqrt{(3)} \times \gamma_{M0})$ = **138.58** kN

Moment resistance at notch $M_{v,N,Rd} = f_{y,b} \times W_{el,N,y} / \gamma_{M0}$ = **9.29** kNm

Design moment at notch $M_{v,Ed} = V_{Ed1} \times (t_p + l_n)$ = **6.75** kNm

Utlilisation $M_{v,Ed} / M_{v,N,Rd}$ = 0.727

PASS - Notch resistance is greater than design force

Check 6: Supported beam - Local stability of notched beam

Single notch

Maximum notch depth $h_b / 2$ = **101.6** mm

Actual notch depth d_{nt} = **50** mm

Maximum notch length ($h_b/t_{w,b}$ <= 54.3) h_b = **203.2** mm

Actual notch length l_n = **125** mm

PASS - Local stability is accounted for

Check 8: Connection - Bolt group

Bolt tensile stress area A_s = **157** mm^2

Bolt shear stress factor α_v = **0.6**

Bolt shear resistance $F_{v,Rd} = \alpha_v \times f_{u,bolt} \times A_s / \gamma_{M2,b}$ = **60.29** kN

For the end plate $\alpha_{b,p} = Min(e_1 / (3 \times d_0), p_1 / (3 \times d_0) - 1/4, f_{u,bolt} / f_{u,p}, 1)$ = **0.46**

$k_{1,p} = Min(2.8 \times e_2 / d_0 - 1.7, 1.4 \times p_3 / d_0 - 1.7, 2.5)$ = **2.19**

For the supporting member $\alpha_{b,2} = Min(p_1 / (3 \times d_0) - 1/4, f_{u,bolt} / f_u, 1)$ = **1**

$k_{1,2} = Min(1.4 \times p_3 / d_0 - 1.7, 2.5)$ = **2.5**

Bearing on the end plate $F_{b,Rd,p} = k_{1,p} \times \alpha_{b,p} \times f_{u,p} \times d_b \times t_p / \gamma_{M2,b}$ = **53.18** kN

Bearing on the supporting member $F_{b,Rd,2} = k_{1,2} \times \alpha_{b,2} \times f_u \times d_b \times t_w / \gamma_{M2,b}$ = **112.83** kN

Minimum bearing resistance $F_{b,Rd1} = Min(F_{b,Rd,p}, F_{b,Rd,2})$ = **53.18** kN

Resistance of the bolt group $F_{Rd} = 0.8 \times n_b \times F_{v,Rd}$ = **192.92** kN

Utilisation V_{Ed1} / F_{Rd} = **0.259**

PASS - Bolt group resistance is greater than design force

Check 9: Connection - End plate in shear

Net shear area $A_{v,net} = t_p \times (h_p - n_{1,1} \times d_0)$ = **890** mm^2

Edge shear area $A_{nt} = t_p \times (e_2 - d_0 / 2)$ = **160** mm^2

Shear area from end bolt $A_{nv} = t_p \times (h_p - e_1 - (n_{1,1} - 0.5) \times d_0)$ = **730** mm^2

Gross section shear resistance $V_{Rd,g} = (2 \times h_p \times t_p) / 1.27 \times f_{y,p} / (\sqrt{(3)} \times \gamma_{M0})$ = **312.54** kN

Net section shear resistance $V_{Rd,n} = 2 \times A_{v,net,A\_c1} \times f_{u,plate,A\_c1} / (\sqrt{(3)} \times \gamma_{M2,c})$ = **383.05** kN

Block tearing resistance $V_{Rd,b} = 2 \times (f_{u,p} \times A_{nt} / \gamma_{M2,c} + f_{y,p} \times A_{nv} / (\sqrt{(3)} \times \gamma_{M0}))$ = **351.08** kN

End plate in-plane bending resistance $h_p < 1.36p_3$ - No additional requirements

End plate shear resistance $V_{Rd,pl,min} = Min(V_{Rd,g}, V_{Rd,n}, V_{Rd,b})$ = **312.54** kN

Utilisation _PlateShearUtilisationA_c1 = **0.16**

PASS - Shear resistance of end plate greater than design force

Check 10: Supporting beam - Shear

Distance from top bolt to flange — $e_{1,t}$ = **75** mm

Distance from bottom bolt to flange — $e_{1,b}$ = **104** mm

Minimum top distance — $e_t = \text{Min}(e_{1,t}, 5 \times d_b)$ = **75** mm

Minimum bottom distance — $e_b = \text{Min}(e_{1,b}, p_3 / 2, 5 \times d_b)$ = **45** mm

Shear area of supporting member — $A_v = t_w \times (e_t + (n_{1,1} - 1) \times p_1 + e_b)$ = **1677** mm^2

Net shear area of supporting member — $A_{v,net} = A_v - n_{1,1} \times d_0 \times t_w$ = **1367** mm^2

Local shear resistance — $V_{Rd,min} = \text{Min}(A_v \times f_y / (\sqrt{(3)} \times \gamma_{M0}), A_{v,net} \times f_u / (\sqrt{(3)} \times \gamma_{M2,c}))$ = **266.26** kN

Utilisation — $V_{Ed1}/2 / V_{Rd,min}$ = **0.094**

PASS - Beam shear resistance is greater than design force

Check 11: Tying resistance - Plate and bolts

Effective end distance — $e_{1A} = \text{Min}(e_1, 0.5 \times (p_3 - t_{w,b} - 2 \times a_w \times \sqrt{(2)}) + d_0/2)$ = **25** mm

Effective bolt pitch — $p_{1A} = \text{Min}(p_1, p_3 - t_{w,b} - 2 \times a_w \times \sqrt{(2)} + d_0)$ = **75** mm

Minimum end distance — $e_{min} = e_2$ = **25** mm

Bolt factor — k_2 = **0.9**

Distance from weld throat to bolt — $m_w = (p_3 - t_{w,b} - 2 \times 0.8 \times a_w \times \sqrt{(2)}) / 2$ = **36** mm

$n_w = \text{Min}(e_{min}, 1.25 \times m_w)$ = **25** mm

Width across bolt head points — d_w = **26** mm

$e_w = d_w / 4$ = **6.5** mm

Effective length of equivalent T-stub — $\Sigma l_{eff} = 2 \times e_{1A} + (n_{1,1} - 1) \times p_{1A}$ = **125.0** mm

Moment resistance of plate — $M_{pl,1,Rd,u} = (0.25 \times \Sigma l_{eff} \times t_p^2 \times f_{u,p}) / \gamma_{M,u}$ = **1.16** kNm

$M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$ = **1.16** kNm

Mode 1 plate failure — $F_{Rd,u,1} = (8 \times n_w - 2 \times e_w) \times M_{pl,1,Rd,u} / (2 \times m_w \times n_w - e_w \times (m_w + n_w))$ = **155.36** kN

Individual bolt resistance — $F_{t,Rd,u} = k_2 \times f_{u,bolt} \times A_s / \gamma_{M,u}$ = **102.76** kN

Group bolt resistance — $\Sigma F_{t,Rd,u} = n_b \times F_{t,Rd,u}$ = **411.05** kN

Mode 2 bolt and plate failure — $F_{Rd,u,2} = (2 \times M_{pl,2,Rd,u} + n_w \times \Sigma F_{t,Rd,u}) / (m_w + n_w)$ = **206.78** kN

Mode 3 bolt failure — $F_{Rd,u,3} = \Sigma F_{t,Rd,u}$ = **411.05** kN

Minimum resistance — $F_{Rd,u,min} = \text{Min}(F_{Rd,u,1}, F_{Rd,u,2}, F_{Rd,u,3})$ = **155.36** kN

Utilisation — $F_{Ed1} / F_{Rd,u,min}$ = **0.161**

PASS - Tying resistance of plate and bolts is greater than design force

Check 12: Tying resistance - Supported beam web

Web resistance — $F_{Rd,u} = (t_{w,b} \times h_p \times f_{u,b}) / \gamma_{M,u}$ = **251.59** kN

Utilisation — $F_{Ed1} / F_{Rd,u}$ = **0.099**

PASS - Supported beam web tying resistance is greater than design force

STEEL BEAM SPLICE DESIGN (EN1993)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009

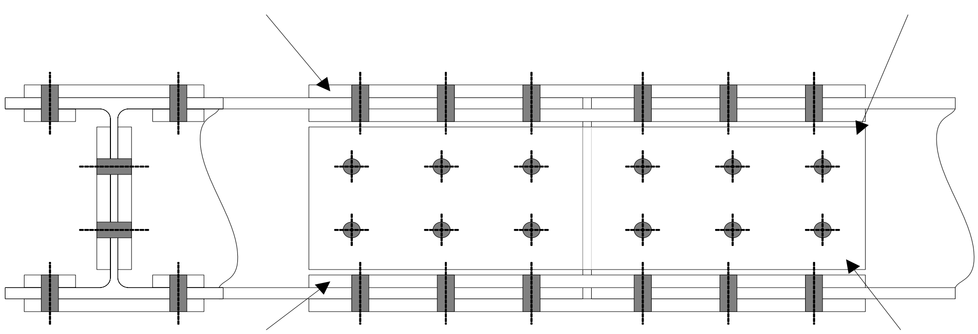
Tedds calculation version 1.0.00

Design summary

Overall design status PASS

Overall design utilisation 0.694

Description	Unit	Design	Resistance	Utilisation	Result
Flange bolt group	kN	104.9	188.2	0.558	PASS
Flange bolt slip resist. SLS	kN	86.5	124.7	0.694	PASS
Flange plate compression	kN	629.6	1452.0	0.434	PASS
Flange plate tension flange	kN	530.6	1298.9	0.409	PASS
Flange plate block tearing	kN	530.6	2536.3	0.209	PASS
Flange compression	kN	629.6	994.2	0.633	PASS
Flange tension	kN	530.6	994.2	0.534	PASS
Web bolt group	kN	88.2	128.2	0.688	PASS
Web bolt slip resist. SLS	kN	66.1	124.7	0.530	PASS
Web plate shear	kN	125.0	720.1	0.174	PASS
Web plate block tearing	kN	125.0	958.0	0.130	PASS
Web plate combined				0.692	PASS
Beam web net shear	kN	125.0	470.0	0.266	PASS



Cover plate Splice

Design forces

Design moment (ULS) M_{Ed} = **150.0** kNm

Design moment (SLS) $M_{Ed,ser}$ = **125.0** kNm

Design shear force (ULS) V_{Ed} = **125.0** kN

Design shear force (SLS) $V_{Ed,ser}$ = **90.0** kN

Design axial force (ULS) N_{Ed} = **125.0** kN

Design axial force (SLS) $N_{Ed,ser}$ = **90.0** kN

Partial factors

Resistance of cross section γ_{M0} = **1.00**

Resistance of members to instability γ_{M1} = **1.00**

Cross sections in tension to fracture γ_{M2} = **1.10**

Resistance of bolts $\gamma_{M2,b}$ = **1.25**

Slip resistance γ_{M3} = **1.25**

Slip resistance, servicability $\gamma_{M3,ser}$ = **1.10**

Structural integrity $\gamma_{M,u}$ = **1.10**

Beam details

Section name UC 254x254x73

Steel grade S275

Yield strength $f_{y,bm}$ = **275** N/mm^2

Ultimate strength $f_{u,bm}$ = **410** N/mm^2

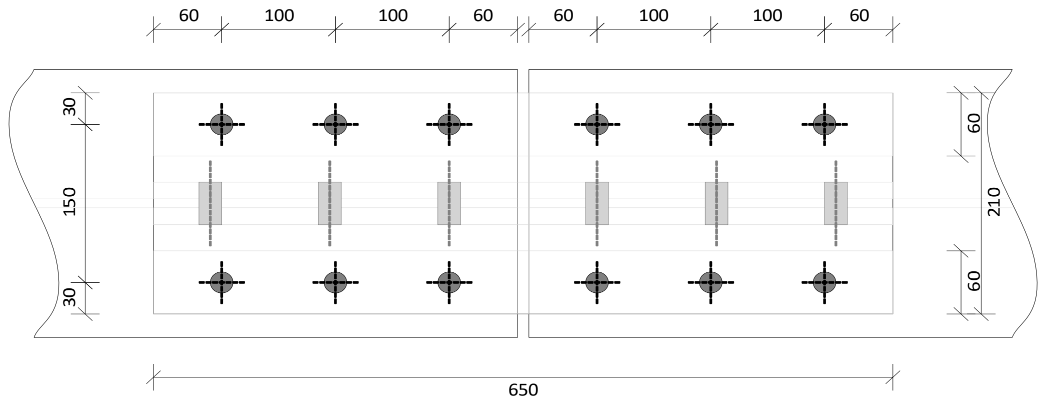
Section height h = **254.1** mm

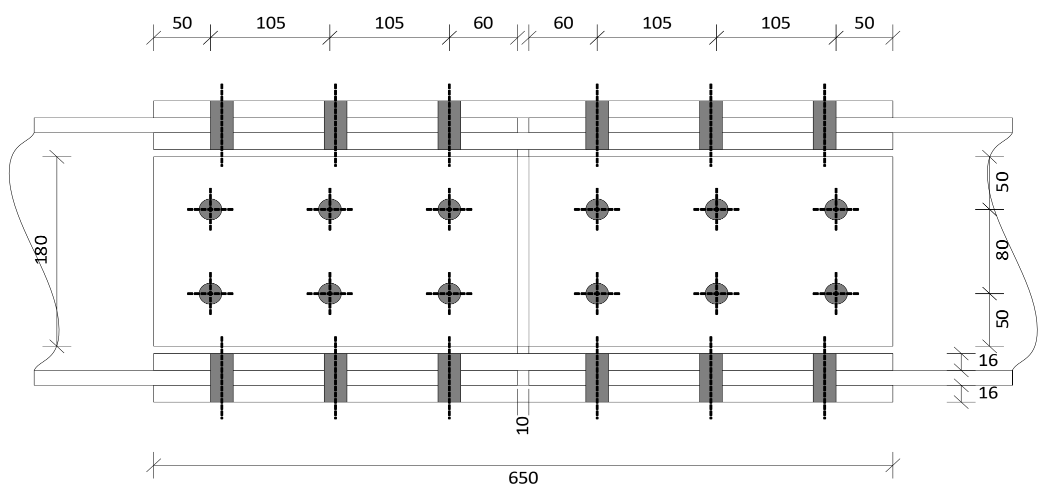
Section width b = **254.6** mm

Flange thickness t_f = **14.2** mm

Web thickness t_w = **8.6** mm

Gap between beams g_v = **10.0** mm





Flange bolt details

Number of bolt rows $n_{1,fp}$ = **3**

Bolt size **M20**

Bolt grade **8.8**

Yield strength $f_{yb,fp}$ = **640** N/mm^2

Ultimate strength $f_{ub,fp}$ = **800** N/mm^2

End distance in plate $e_{1,fp}$ = **60** mm

End distance in beam	$e_{1,cf}$ = **60** mm
Edge distance in plate	$e_{2,fp}$ = **30** mm
Bolt pitch	$p_{1,fp}$ = **100** mm
Bolt gauge	$p_{2,fp}$ = **150** mm

Flange plate details

Internal plate width	$b_{fp,int} = 2 \times e_{2,fp}$ = **60** mm
External plate width	$b_{fp,ext} = 2 \times e_{2,fp} + p_{2,fp}$ = **210** mm
Plate length	$l_{fp} = g_v + 2 \times (n_{1,fp} - 1) \times p_{1,fp} + 2 \times (e_{1,cf} + e_{1,fp})$ = **650** mm
Plate thickness	t_{fp} = **16** mm
Plate grade	**S275**
Yield strength	$f_{y,fp}$ = **275** N/mm^2
Ultimate strength	$f_{u,fp}$ = **410** N/mm^2

Web bolt details

Number of bolt columns	$n_{1,wp}$ = **3**
Number of bolt rows	$n_{2,wp}$ = **2**
Bolt size	**M20**
Bolt grade	**8.8**
Yield strength	$f_{yb,wp}$ = **640** N/mm^2
Ultimate strength	$f_{ub,wp}$ = **800** N/mm^2
End distance in plate	$e_{1,wp}$ = **50** mm
End distance in beam	$e_{1,cw}$ = **60** mm
Edge distance in plate	$e_{2,wp}$ = **50** mm
Bolt gauge	$p_{1,wp}$ = **105** mm
Bolt pitch	$p_{2,wp}$ = **80** mm

Web plate details

Number of web plates	N_{wp} = **2**
Plate width	$b_{wp} = g_v + 2 \times (n_{1,wp} - 1) \times p_{1,wp} + 2 \times (e_{1,cw} + e_{1,wp})$ = **650** mm
Plate height	$h_{wp} = (n_{2,wp} - 1) \times p_{2,wp} + 2 \times e_{2,wp}$ = **180** mm
Plate thickness	t_{wp} = **16** mm
Plate grade	**S275**
Yield strength	$f_{y,wp}$ = **275** N/mm^2
Ultimate strength	$f_{u,wp}$ = **410** N/mm^2

Continuity - EN1993-1-8 cl.6.2.7.1(13)

Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = W_{pl.y} \times f_{y,bm} / \gamma_{M0}$ = **272.8** kNm
Min design bending resist. momnt for continuity	$M_{Ed,cont} = 0.25 \times M_{c,y,Rd}$ = **68.2** kNm
	$M_{Ed,cont} / M_{Ed}$ = **0.455**

M_{Ed} is greater or equal to 0.25 × $M_{c,y,Rd}$ so continuity is satisfied

Internal forces - ULS

Second moment of area	$I_{y,web} = (h - 2 \times t_f)^3 \times t_w / 12$ = **824** cm^4
Area of web	$A_w = (h - 2 \times t_f) \times t_w$ = **1941** mm^2
Force in each flange due to moment	$F_{f,M} = (1 - (I_{y,web} / I_y)) \times M_{Ed} / (h - t_f)$ = **580.1** kN
Force in each flange due to axial force	$F_{f,N} = (1 - A_w / A) \times N_{Ed} / 2$ = **49.5** kN
Total force in tension flange	$F_{tf} = F_{f,M} - F_{f,N}$ = **530.6** kN
Total force in compression flange	$F_{cf} = F_{f,M} + F_{f,N}$ = **629.6** kN

Flange bolt forces - ULS

Force in compression flange bolts — $F_{cf,v} = F_{cf} / (2 \times n_{1,fp})$ = **104.9** kN

Force in tension flange bolts — $F_{tf,v} = F_{tf} / (2 \times n_{1,fp})$ = **88.4** kN

Maximum design force on flange bolts — $F_{f,v,Ed} = \max(F_{cf,v}, F_{tf,v})$ = **104.9** kN

Flange bolt resistance (Table 3.4)

Flange plate bearing strength — $f_{u,fp} \times 2 \times t_{fp}$ = **13120** kN/m

Beam flange bearing strength — $f_{u,bm} \times t_f$ = **5822** kN/m

Critical section for bearing — Beam flange

Bolt shear area — $A_{s,fp}$ = **245.0** mm^2

Shear area factor — $\alpha_{v,fp}$ = **0.6**

Length of joint — $L_j = (n_{1,fp} - 1) \times p_{1,fp}$ = **200** mm

Reduction factor due to length of joint — $\beta_{LF,fp}$ = **1.000**

Shear resistance of single bolt — $F_{v,fp,Rd} = \alpha_{v,fp} \times f_{ub,fp} \times A_{s,fp} / \gamma_{M2,b}$ = **94.1** kN

Bolt bearing factors — $k_{1,f} = \min(1.4 \times p_{2,fp} / d_{0,fp} - 1.7, 2.5)$ = **2.50**

$\alpha_{b,f} = \min(e_{1,cf} / (3 \times d_{0,fp}), p_{1,fp} / (3 \times d_{0,fp}) - 1/4, f_{ub,fp} / f_{u,bm}, 1.0)$ = **0.91**

Bearing resistance of bolt — $F_{b,f,Rd} = k_{1,f} \times \alpha_{b,f} \times f_{u,bm} \times d_{b,fp} \times t_f / \gamma_{M2,b}$ = **211.7** kN

Design resistance of bolt — $F_{f,Rd} = \min(2 \times F_{v,fp,Rd}, F_{b,f,Rd})$ = **188.2** kN

$F_{f,v,Ed} / F_{f,Rd}$ = **0.558**

PASS - Bolt resistance exceeds applied force on bolt

Internal forces - SLS

Force in each flange due to moment — $F_{f,M,ser} = (1 - (I_{y,web} / I_y)) \times M_{Ed,ser} / (h - t_f)$ = **483.4** kN

Force in each flange due to axial force — $F_{f,N,ser} = (1 - A_w / A) \times N_{Ed,ser} / 2$ = **35.6** kN

Total force in tension flange — $F_{tf,ser} = F_{f,M,ser} - F_{f,N,ser}$ = **447.8** kN

Total force in compression flange — $F_{cf,ser} = F_{f,M,ser} + F_{f,N,ser}$ = **519.0** kN

Flange bolt forces - SLS

Force in compression flange bolts — $F_{cf,v,ser} = F_{cf,ser} / (2 \times n_{1,fp})$ = **86.5** kN

Force in tension flange bolts — $F_{tf,v,ser} = F_{tf,ser} / (2 \times n_{1,fp})$ = **74.6** kN

Maximum design force on flange bolts — $F_{f,v,Ed,ser} = \max(F_{cf,v,ser}, F_{tf,v,ser})$ = **86.5** kN

Flange bolt resistance SLS

Pre-loading force — $F_{p,C,fp,ser} = 0.7 \times f_{ub,fp} \times A_{s,fp}$ = **137.2** kN

Bolt type factor — k_s = **1.00**

Slip factor — μ = **0.50**

Slip resistance of single bolt — $F_{s,fp,Rd,ser} = k_s \times 2 \times \mu \times F_{p,C,fp,ser} / \gamma_{M3,ser}$ = **124.7** kN

$F_{f,v,Ed,ser} / F_{s,fp,Rd,ser}$ = **0.694**

PASS - Bolt resistance exceeds applied force on bolt

Flange cover plates - resistance of plates in compression

Gross area of plate to 1 flange — $A_{fp} = b_{fp,ext} \times t_{fp} + 2 \times b_{fp,int} \times t_{fp}$ = **5280** mm^2

Pitch between the bolts rows of the splice — $p_{1,j} = \max(2 \times e_{1,cf} + g_v, p_{1,fp})$ = **130** mm

Critical buckling length — $L_{cr} = 0.6 \times p_{1,j}$ = **78** mm

Steel strength factor (cl.6.3.1.3) — $\varepsilon = \sqrt{(235\text{N/mm}^2 / f_{y,fp})}$ = **0.92**

Radius of gyration of plate — $i_{z,fp} = t_{fp} / \sqrt{(12)}$ = **4.6** mm

Non-dimensional slenderness factor (cl.6.3.1.3) — $\lambda_1 = 93.9 \times \varepsilon$ = **86.80**

$\bar{\lambda} = L_{cr} / i_{z,fp} \times 1 / \lambda_1$ = **0.19**

Buckling curve - solid section (Table 6.2) — c

Imperfection factor (Table 6.1) α = **0.49**

Buckling reduction factor (cl.6.3.1.2) $\Phi = 0.5 \times (1 + \alpha \times (\bar{\lambda} - 0.2) + \bar{\lambda}^2) =$ **0.52**

$\chi_{fp} = 1.0 =$ **1.00**

Design compressive resistance (eqn. 6.47) $N_{b,fp,Rd} = \chi_{fp} \times A_{fp} \times f_{y,fp} / \gamma_{M1} =$ **1452** kN

Utilisation $F_{cf} / N_{b,fp,Rd} =$ **0.434**

PASS - Flange cover plate compressive resistance greater than design compressive force

Flange cover plates - resistance of plates in tension

Net area of plate to 1 flange $A_{fp,net} = A_{fp} - 4 \times t_{fp} \times d_{0,fp} =$ **3872** mm^2

Tension resistance of gross area (eqn.6.6) $N_{pl,fp,Rd} = A_{fp} \times f_{y,fp} / \gamma_{M0} =$ **1452** kN

Tension resistance of net area (eqn.6.6) $N_{u,fp,Rd} = 0.9 \times A_{fp,net} \times f_{u,fp} / \gamma_{M2} =$ **1298.88** kN

Design tension resistance $N_{t,fp,Rd} = \min(N_{pl,fp,Rd}, N_{u,fp,Rd}) =$ **1298.88** kN

Utilisation $F_{tf} / N_{t,fp,Rd} =$ **0.409**

PASS - Flange cover plate tensile resistance greater than design tensile force

Flange cover plates - block tearing

Net area of plate to subject to shear $A_{fp,nv} = 4 \times t_{fp} \times (e_{1,fp} + (n_{1,fp} - 1) \times p_{1,fp} - (n_{1,fp} - 0.5) \times d_{0,fp}) =$ **13120** mm^2

Net area of plate to subject to tension $A_{fp,nt} = t_{fp} \times (2 \times e_{2,fp} - d_{0,fp}) + t_{fp} \times (b_{fp,int} - d_{0,fp}) =$ **1216** mm^2

Block tearing resistance (EN1993-1-8 eqn.3.9) $N_{bt,Rd} = f_{u,fp} \times A_{fp,nt} / \gamma_{M2} + f_{y,fp} \times A_{fp,nv} / (\sqrt{(3)} \times \gamma_{M0}) =$ **2536.32** kN

Utilisation $F_{tf} / N_{bt,Rd} =$ **0.209**

PASS - Flange cover plate block tearing resistance greater than design tensile force

Beam flange - resistance in compression

Gross area of flange $A_f = b \times t_f =$ **3615.3** mm^2

Assume χ is 1.0 due to presence of beam web χ = **1.00**

Design compressive resistance (eqn. 6.47) $N_{b,Rd} = \chi \times A_f \times f_{y,bm} / \gamma_{M1} =$ **994.21** kN

Utilisation $F_{cf} / N_{b,Rd} =$ **0.633**

PASS - Beam flange compressive resistance greater than design compressive force

Beam flange - resistance in tension

Net area of flange $A_{f,net} = A_f - 2 \times t_f \times d_{0,fp} =$ **2990.5** mm^2

Tension resistance of gross area (eqn.6.6) $N_{pl,Rd} = A_f \times f_{y,bm} / \gamma_{M0} =$ **994.21** kN

Tension resistance of net area (eqn.6.6) $N_{u,Rd} = 0.9 \times A_{f,net} \times f_{u,bm} / \gamma_{M2} =$ **1003.18** kN

Design tension resistance $N_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) =$ **994.21** kN

Utilisation $F_{tf} / N_{t,Rd} =$ **0.534**

PASS - Beam flange tensile resistance greater than design tensile force

Internal forces - ULS

Moment in the web (at the CL of the splice) $M_w = M_{Ed} \times I_{y,web} / I_y =$ **10.8** kNm

Force in web due to axial force $F_{w,N} = N_{Ed} \times A_w / A =$ **26.1** kN

Force in web due to vertical shear $F_{w,V} = V_{Ed} =$ **125.0** kN

Web bolt forces - ULS

Web bolt group inertia $I_{b,wp} =$ **53700** mm^2

Extreme horiz. bolt position from group centroid $x_{max} = (p_{1,wp} \times (n_{1,wp} - 1)) / 2 =$ **105.0** mm

Extreme vert. bolt position from group centroid $z_{max} = (p_{2,wp} \times (n_{2,wp} - 1)) / 2 =$ **40.0** mm

Number of bolts in bolt group $n_{wp} = n_{1,wp} \times n_{2,wp} =$ **6**

Add. moment due to eccentricity of bolt group $M_{add} = V_{Ed} \times (g_v / 2 + e_{1,cw} + (p_{1,wp} \times (n_{1,wp} - 1)) / 2) =$ **21.3** kNm

Horizontal component of force on extreme bolt $F_{x,M} = (M_w + M_{add}) \times z_{max} / I_{b,wp} =$ **23.9** kN

Vertical component of force on extreme bolt $F_{z,M} = (M_w + M_{add}) \times x_{max} / I_{b,wp} =$ **62.7** kN

Resultant force on extreme bolt — $F_{w,v,Ed} = ((F_{z,M} + F_{w,V} / n_{wp})^2 + (F_{x,M} + F_{w,N} / n_{wp})^2)^{0.5}$ = **88.2** kN

Web bolt resistance (Table 3.4)

Web plate bearing strength — $f_{u,wp} \times N_{wp} \times t_{wp}$ = **13120** kN/m

Beam web bearing strength — $f_{u,bm} \times t_w$ = **3526** kN/m

Critical section for bearing — Beam web

Bolt shear area — $A_{s,wp}$ = **245.0** mm^2

Shear area factor — $\alpha_{v,wp}$ = **0.6**

Shear resistance of single bolt — $F_{v,wp,Rd} = N_{wp} \times \alpha_{v,wp} \times f_{ub,wp} \times A_{s,wp} / \gamma_{M2,b}$ = **188.2** kN

Bolt bearing factors — $k_{1,w} = \min(1.4 \times p_{2,wp} / d_{0,wp} - 1.7, 2.5)$ = **2.50**

$\alpha_{b,w} = \min(e_{1,cw} / (3 \times d_{0,wp}), p_{1,wp} / (3 \times d_{0,wp}) - 1/4, f_{ub,wp} / f_{u,bm}, 1.0)$ = **0.91**

Bearing resistance of bolt — $F_{b,w,Rd} = k_{1,w} \times \alpha_{b,w} \times f_{u,bm} \times d_{b,wp} \times t_w / \gamma_{M2,b}$ = **128.2** kN

Design resistance of bolt — $F_{w,Rd} = \min(F_{v,wp,Rd}, F_{b,w,Rd})$ = **128.2** kN

$F_{w,v,Ed} / F_{w,Rd}$ = **0.688**

PASS - Bolt resistance exceeds applied force on bolt

Internal forces - SLS

Moment in the web (at the CL of the splice) — $M_{w,ser} = M_{Ed,ser} \times I_{y,web} / I_y$ = **9.0** kNm

Force in web due to axial force — $F_{w,N,ser} = N_{Ed,ser} \times A_w / A$ = **18.8** kN

Force in web due to vertical shear — $F_{w,V,ser} = V_{Ed,ser}$ = **90.0** kN

Web bolt forces - SLS

Add. moment due to eccentricity of bolt group — $M_{add,ser} = V_{Ed,ser} \times (g_v / 2 + e_{1,cw} + (p_{1,wp} \times (n_{1,wp} - 1)) / 2)$ = **15.3** kNm

Horizontal component of force on extreme bolt — $F_{x,M,ser} = (M_{w,ser} + M_{add,ser}) \times z_{max} / I_{b,wp}$ = **18.1** kN

Vertical component of force on extreme bolt — $F_{z,M,ser} = (M_{w,ser} + M_{add,ser}) \times x_{max} / I_{b,wp}$ = **47.6** kN

Resultant force on extreme bolt — $F_{w,v,Ed,ser} = ((F_{z,M,ser} + F_{w,V,ser} / n_{wp})^2 + (F_{x,M,ser} + F_{w,N,ser} / n_{wp})^2)^{0.5}$ = **66.1** kN

Web bolt resistance SLS

Pre-loading force — $F_{p,C,wp,ser} = 0.7 \times f_{ub,wp} \times A_{s,wp}$ = **137.2** kN

Slip resistance of single bolt — $F_{s,wp,Rd,ser} = k_s \times N_{wp} \times \mu \times F_{p,C,wp,ser} / \gamma_{M3,ser}$ = **124.7** kN

$F_{w,v,Ed,ser} / F_{s,wp,Rd,ser}$ = **0.530**

PASS - Bolt resistance exceeds applied force on bolt

Resistance of web cover plate in shear

Resist. of gross shear area of web plate (Eqn.6.18) — $V_{wp,g,Rd} = 2 \times h_{wp} \times t_{wp} \times f_{y,wp} / (1.27 \times \sqrt{(3)} \times \gamma_{M0})$ = **720.10** kN

Net area of web plate — $A_{v,wp,net} = (h_{wp} - n_{2,wp} \times d_{0,wp}) \times t_{wp}$ = **2176** mm^2

Resist. of net shear area of web plate (Eqn.6.18) — $V_{wp,net,Rd} = 2 \times A_{v,wp,net} \times f_{u,wp} / (\sqrt{(3)} \times \gamma_{M2})$ = **936.53** kN

Plastic resistance of web cover plate — $V_{pl,wp,Rd} = \min(V_{wp,g,Rd}, V_{wp,net,Rd})$ = **720.10** kN

$V_{Ed} / V_{pl,wp,Rd}$ = **0.174**

PASS - Shear resistance of web plate exceeds applied shear

Block tearing of web cover plate

Area subject to tension — $A_{nt} = t_{wp} \times (e_{1,wp} - d_{0,wp} / 2)$ = **624** mm^2

Area subject to shear — $A_{nv} = t_{wp} \times (h_{wp} - e_{2,wp} - (n_{2,wp} - 0.5) \times d_{0,wp})$ = **1552** mm^2

Block tearing resistance — $V_{b,Rd} = 2 \times (f_{u,wp} \times A_{nt} / \gamma_{M2} + f_{y,wp} \times A_{nv} / (\sqrt{(3)} \times \gamma_{M0}))$ = **957.99** kN

$V_{Ed} / V_{b,Rd}$ = **0.130**

PASS - Block tearing resistance of web plate exceeds applied shear

Resistance of beam web shear accounting for holes

Height of web — $h_w = h - 2 \times t_f$ = **225.7** mm

Summer House, Upper Court Road
Woldingham SURREY CR3 7BF
support@planningapplications.com 07922 148 701

η = **1.000**

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **2562** mm²

Net shear area — $A_{v,net} = A_v - (n_{2,wp} \times d_{0,wp} \times t_w)$ = **2184** mm²

Design shear net resistance - cl 6.2.6(2) — $V_{nw,Rd} = A_{v,net} \times f_{u,bm} / (\sqrt{(3)} \times \gamma_{M2})$ = **470.0** kN

$V_{Ed} / V_{nw,Rd}$ = **0.266**

PASS - Net shear resistance of beam web exceeds applied shear force

Resistance of web cover plate to combined bending, shear and axial

Elastic modulus of web cover plate — $W_{wp} = t_{wp} \times h_{wp}^2 / 6$ = **86400** mm³

Bending resistance of web cover plate — $M_{c,wp,Rd} = 2 \times W_{wp} \times f_{y,wp} / \gamma_{M0}$ = **47.5** kNm

Axial resistance of web cover plate — $N_{wp,Rd} = 2 \times t_{wp} \times h_{wp} \times f_{y,wp}$ = **1584.0** kN

Appied moment — $M_{wp,Ed} = M_w + M_{add}$ = **32.1** kNm

Applied axial force — $N_{wp,Ed} = F_{w,N}$ = **26.1** kN

Interaction formula — $N_{wp,Ed} / N_{wp,Rd} + M_{wp,Ed} / M_{c,wp,Rd}$ = **0.692**

PASS - Combined bending, shear and axial check satisfied