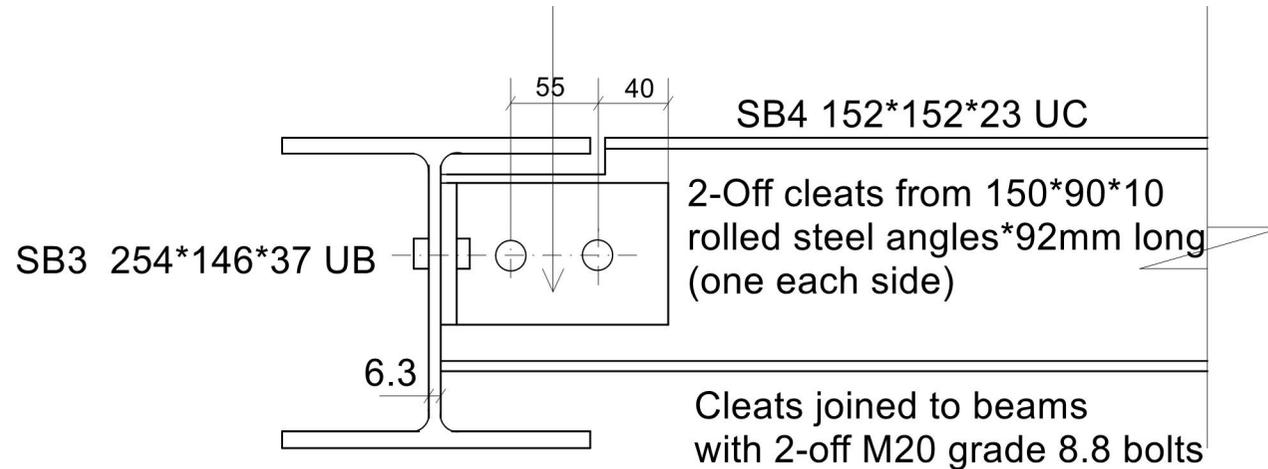


Connections between 152*152*23 UC and 254*146*37 UB

maximum factored load 25 KN



As an Example : Bearing capacity of M20 bolts passing through 5.8mm thick steel = 48.8 KN

2-off 48.8 KN = 97.6 KN therefore OK.

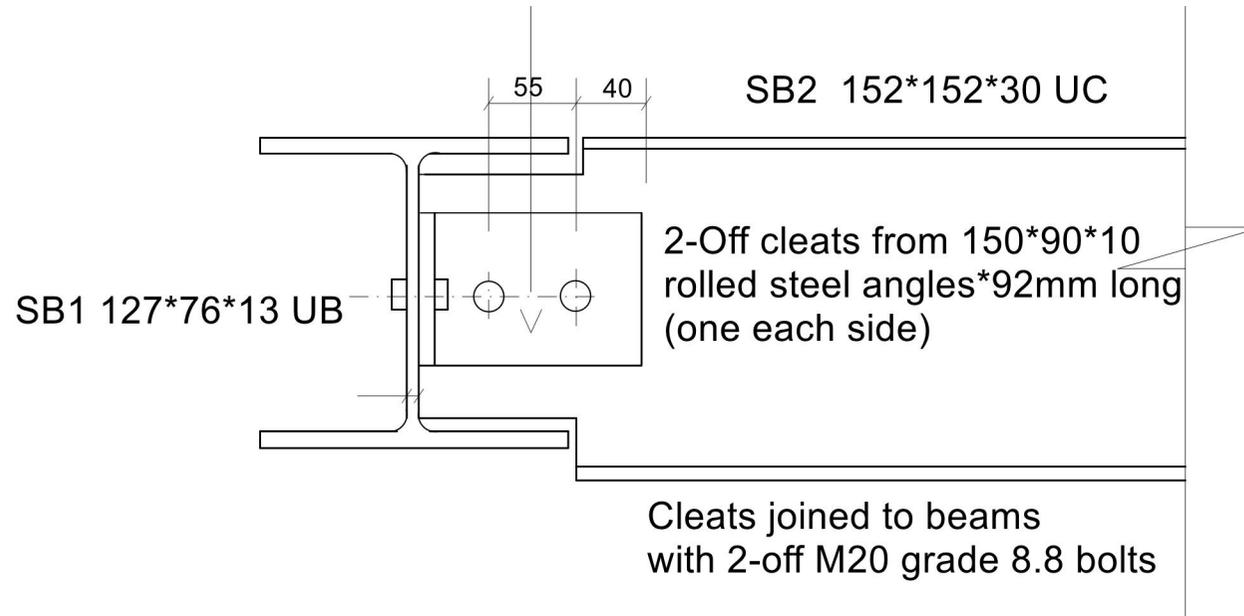
Shearing Resistance of M20 grade 8,8 bolts= 94.1 KN

2-off 94.1 KN = 188.2 KN > 25 KN therefore OK.

Values from P363 Steel building design data
in accordance with Eurocodes and UK National Annexes

Connections between 152*152*30 UC and 127*76*13 UB

maximum factored load 25 KN



As an Example : Bearing capacity of M20 bolts passing through 5.8mm thick steel = 48.8 KN

2-off 48.8 KN = 97.6 KN therefore OK.

Shearing Resistance of M20 grade 8,8 bolts= 94.1 KN

2-off 94.1 KN = 188.2 KN > 25 KN therefore OK.

Values from P363 Steel building design data
in accordance with Eurocodes and UK National Annexes

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Steel Beam Design

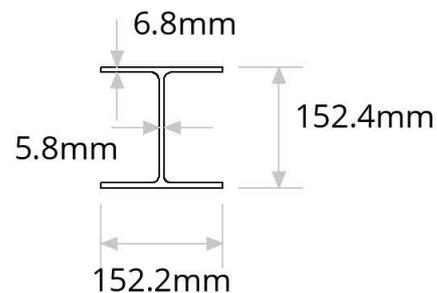
To Eurocode BS EN 1993-1-1/NA:2008

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	158 kN	52.9 kN	34 %	OK
Bending moment	45.1 kNm	35.7 kNm	79 %	OK
Buckling	40.2 kNm	35.7 kNm	89 %	OK
Total deflection	13.5 mm	7.5 mm	56 %	OK
Deflection due to variable actions	7.5 mm	0.9 mm	12 %	OK

Section details

Type	Universal column
Section	152 x 152 x 23 UC
Steel grade	S275
Width	$b = 152 \text{ mm}$
Depth	$h = 152 \text{ mm}$
Web thickness	$t_w = 5.8 \text{ mm}$
Flange thickness	$t_f = 6.8 \text{ mm}$
Root radius	$r = 7.6 \text{ mm}$
Mass per metre	$w = 23 \text{ kg/m}$



Span and restraints

Effective span	$L = 2,700 \text{ mm}$
Buckling length	$L_{cr} = 2,700 \text{ mm}$

Deflection limits

Variable action deflection limit	$\Delta_Q = L / 360 = 7.49 \text{ mm}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = 13.5 \text{ mm}$

Safety factors

Partial factor for permanent actions	$\gamma_G = 1.35$
Partial factor for variable actions	$\gamma_Q = 1.5$

Loading details



Self weight

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Permanent action $SW = w \times 9.81 / 1000 = \mathbf{0.226 \text{ kN/m}}$



Load 1: UDL - Sloping roof, 30° to 45°

Permanent action $G_1 = \mathbf{1.41 \text{ kN/m}^2} \times \mathbf{3.57 \text{ m}} = \mathbf{5.04 \text{ kN/m}}$

Variable action $Q_1 = \mathbf{0.75 \text{ kN/m}^2} \times \mathbf{3.57 \text{ m}} = \mathbf{2.68 \text{ kN/m}}$



Load 2: UDL - Ceiling beneath sloping roof

Permanent action $G_2 = \mathbf{0.3 \text{ kN/m}^2} \times \mathbf{3.57 \text{ m}} = \mathbf{1.07 \text{ kN/m}}$

Variable action $Q_2 = \mathbf{0.25 \text{ kN/m}^2} \times \mathbf{3.57 \text{ m}} = \mathbf{0.893 \text{ kN/m}}$



Load 3: UDL - 225mm Brickwork + Plaster or render on ONE side

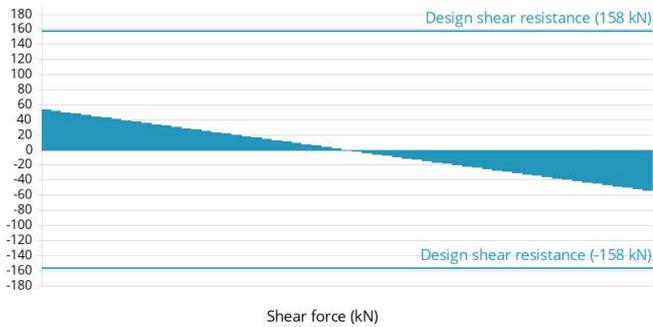
Permanent action $G_3 = \mathbf{4.7 \text{ kN/m}^2} \times \mathbf{4 \text{ m}} = \mathbf{18.8 \text{ kN/m}}$

Variable action $Q_3 = \mathbf{0 \text{ kN/m}^2} \times \mathbf{4 \text{ m}} = \mathbf{0 \text{ kN/m}}$

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	33.9 kN	4.81 kN	38.7 kN	52.9 kN
Right reaction	33.9 kN	4.81 kN	38.7 kN	52.9 kN

Design shear force

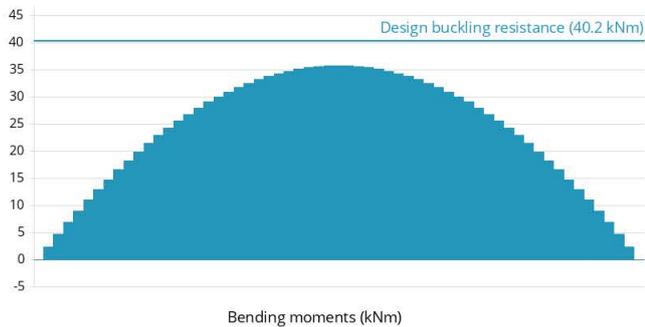


Design shear force $V_{Ed} = \mathbf{52.9 \text{ kN}}$
 Design shear resistance $V_{c,Rd} = \mathbf{158 \text{ kN}}$
 Utilisation $V_{Ed} / V_{c,Rd} = \mathbf{34 \%}$

OK

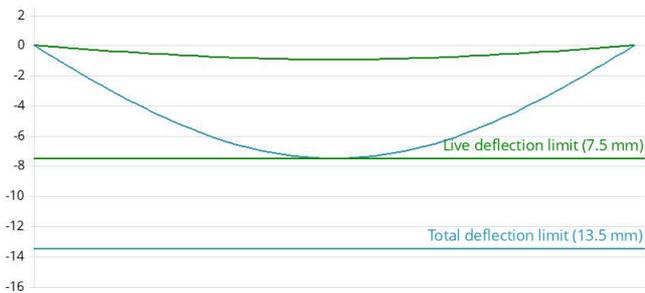
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Design bending moment



Design bending moment, major axis	$M_{Ed} = 35.7$ kNm
Design resistance for bending	$M_{c,Rd} = 45.1$ kNm
Bending utilisation	$M_{Ed} / M_{c,Rd} = 79$ % OK
Design resistance for buckling	$M_{b,Rd} = 40.2$ kNm
Buckling utilisation	$M_{Ed} / M_{b,Rd} = 89$ % OK

Deflection



Variable action deflection limit	$\Delta_Q = 7.5$ mm
Variable action deflection	$\delta_Q = 0.9$ mm OK
Total deflection limit	$\Delta_{G+Q} = 13.5$ mm
Total deflection	$\delta_{G+Q} = 7.5$ mm OK

Section properties

Elastic modulus - major axis, yy	$W_{el} = 164$ cm ³
Plastic modulus - major axis, yy	$W_{pl} = 182$ cm ³
Second moment of area - major axis, yy	$I_y = 1,250$ cm ⁴
Second moment of area - minor axis, zz	$I_z = 400$ cm ⁴
Warping constant	$I_w = 0.021$ dm ⁶
Torsional constant	$I_T = 4.63$ cm ⁴
Area of section	$A = 2,920$ 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	$\nu = 0.3$	cl.3.2.6
Shear modulus	$G_s = 81,000$ N/mm ²	cl.3.2.6
Partial factor for resistance of cross-sections	$\gamma_{M0} = 1$	cl.6.1(1)B / BS-EN NA
Partial factor for resistance to instability	$\gamma_{M1} = 1$	cl.6.1(1)B / BS-EN NA
Factor for shear area	$\eta = 1$	EN 1993-1-5:2006 cl.5.1(2) / BS-EN NA
Limiting non dimensional slenderness ratio	$\bar{\lambda}_{LT,\phi} = 0.4$	cl.6.3.2.3(1) / BS-EN NA
Beta factor for buckling reduction factor calculation	$\beta = 0.75$	cl.6.3.2.3(1) / BS-EN NA

Yield strength

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Nominal yield strength for S275 grade and nominal section thickness 6.80 mm

$$f_y = 275 \text{ N/mm}^2$$

Tata blue book

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

Epsilon	$\epsilon = 0.924$	EN 1993-1-1:2005 Table 5.2
Flange ratio for local buckling	$c_f / t_f = 9.65$	
Flange ratio limit for class 1	$9 \epsilon = 8.32$	Table 5.2 (sheet 2 of 3)
Flange ratio limit for class 2	$10 \epsilon = 9.24$	Table 5.2 (sheet 2 of 3)
Flange ratio limit for class 3	$14 \epsilon = 12.9$	Table 5.2 (sheet 2 of 3)
Flange class	$Class_f = 3$	
Web ratio for local buckling	$c_w / t_w = 21.3$	
Web ratio limit for class 1	$72 \epsilon = 66.6$	Table 5.2 (sheet 1 of 3)
Web class	$Class_w = 1$	
Section class	$Class = 3$	

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

Height of web	$h_w = 139 \text{ mm}$	
Shear area for I and H sections	$A_v = 993 \text{ mm}^2$	cl.6.2.6 (3)
Design shear resistance	$V_{pl,Rd} = 158 \text{ kN}$	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 $(h_w / t_w) > (72 \epsilon / \eta)$

Web ratio for shear buckling	$h_w / t_w = 23.9$	EN 1993-1-5:2006 cl.5.1 (2)
Shear buckling limit	$72 \epsilon / \eta = 66.6$	EN 1993-1-5:2006 cl.5.1 (2)
$(h_w / t_w) \leq (72 \epsilon / \eta)$ therefore shear buckling calculation not required		

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

The shear force (53 kN) is less than half of the plastic shear resistance ($158 \text{ kN} / 2 = 79 \text{ kN}$), therefore its effect on moment resistance may be neglected.

Class 3 section, therefore use elastic modulus	$W_{el} = 164,000 \text{ mm}^3$	
Design bending resistance	$M_{c,Rd} = 45.1 \text{ kNm}$	eq (6.13)

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

C1 factor	$C1 = 1$	
Shear modulus of elasticity	$G_s = 81,000 \text{ N/mm}^2$	cl.3.2.6 (1)
Buckling length	$L_{cr} = 2,700 \text{ mm}$	
Critical buckling moment	$M_{cr} = 105 \text{ kNm}$	NCCI SN003b-EN-EU
Class 3 section, therefore use elastic modulus	$W_{el} = 164,000 \text{ mm}^3$	cl.6.3.2.1(3)
Non-dimensional slenderness ratio	$\bar{\lambda}_{LT} = 0.654$	cl.6.3.2.2 (1)
Depth to width ratio for buckling curve	$h / b = 1$	
Buckling curve for h / b ratio	Buckling curve = b	Table 6.5 / BS-EN NA
Imperfection factor for buckling curve b	$\alpha_{LT} = 0.34$	Table 6.3 / BS-EN NA
Intermediate factor for reduction factor calculation	$\phi_{LT} = 0.704$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = 0.892$	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)

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Modified buckling reduction factor $\chi_{LT,mod} = \mathbf{0.892}$ eq (6.58)

Design buckling resistance $M_{b,Rd} = \mathbf{40.2}$ 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)