

	Project	Project ref
	Calcs for	Date

## Steel Beam Design - Structural Ridge Beam 1

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

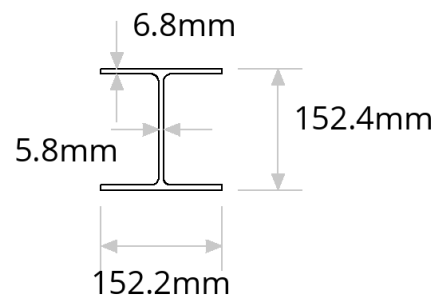
\* You can add your own text, diagrams and photos here \*

### Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	<b>158 kN</b>	<b>21.3 kN</b>	<b>13 %</b>	<b>OK</b>
Bending moment	<b>45.1 kNm</b>	<b>26.6 kNm</b>	<b>59 %</b>	<b>OK</b>
Buckling	<b>30.8 kNm</b>	<b>26.6 kNm</b>	<b>86 %</b>	<b>OK</b>
Total deflection	<b>25 mm</b>	<b>18.8 mm</b>	<b>75 %</b>	<b>OK</b>
Deflection due to variable actions	<b>13.9 mm</b>	<b>7 mm</b>	<b>50 %</b>	<b>OK</b>

### Section details

Type	<b>Universal column</b>
Section	<b>152 x 152 x 23 UC</b>
Steel grade	<b>S275</b>
Width	b = <b>152 mm</b>
Depth	h = <b>152 mm</b>
Web thickness	t <sub>w</sub> = <b>5.8 mm</b>
Flange thickness	t <sub>f</sub> = <b>6.8 mm</b>
Root radius	r = <b>7.6 mm</b>
Mass per metre	w = <b>23 kg/m</b>



### Span and restraints

Effective span	L = <b>5,000 mm</b>
Buckling length	L <sub>cr</sub> = <b>5,000 mm</b>

### Deflection limits

Variable action deflection limit	$\Delta_Q = L / 360 = \mathbf{13.9 mm}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = \mathbf{25 mm}$

### Safety factors

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

### Loading details

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### Self weight

Permanent action

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



### Load 1: UDL - Flat roof, with no permanent access

Permanent action

$$G_1 = 1 \text{ kN/m}^2 \times 1.6 \text{ m} = \mathbf{1.6 \text{ kN/m}}$$

Variable action

$$Q_1 = 0.75 \text{ kN/m}^2 \times 1.6 \text{ m} = \mathbf{1.2 \text{ kN/m}}$$



### Load 2: UDL - Sloping roof, 30° to 45°

Permanent action

$$G_2 = 1.41 \text{ kN/m}^2 \times 1.4 \text{ m} = \mathbf{1.97 \text{ kN/m}}$$

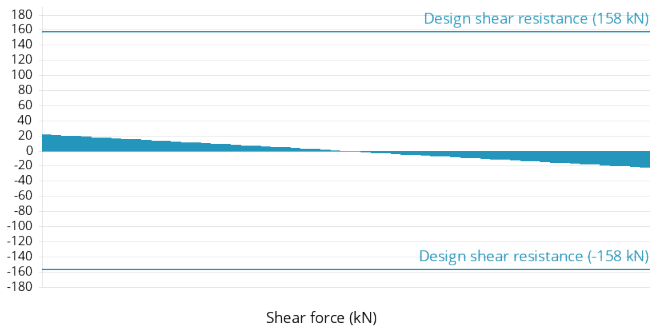
Variable action

$$Q_2 = 0.75 \text{ kN/m}^2 \times 1.4 \text{ m} = \mathbf{1.05 \text{ kN/m}}$$

## Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	<b>9.5 kN</b>	<b>5.62 kN</b>	<b>15.1 kN</b>	<b>21.3 kN</b>
Right reaction	<b>9.5 kN</b>	<b>5.62 kN</b>	<b>15.1 kN</b>	<b>21.3 kN</b>

## Design shear force



Design shear force

$$V_{Ed} = \mathbf{21.3 \text{ kN}}$$

Design shear resistance

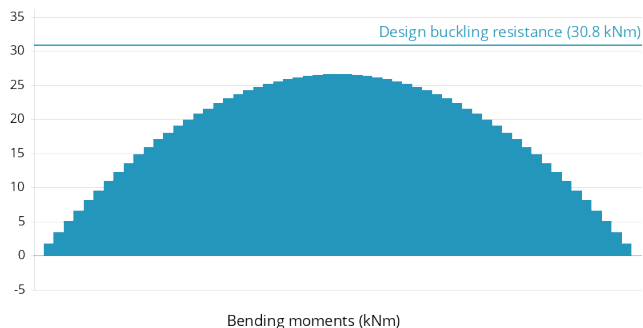
$$V_{c,Rd} = \mathbf{158 \text{ kN}}$$

Utilisation

$$V_{Ed} / V_{c,Rd} = \mathbf{13 \%}$$

**OK**

## Design bending moment



Design bending moment, major axis

$$M_{Ed} = \mathbf{26.6 \text{ kNm}}$$

Design resistance for bending

$$M_{c,Rd} = \mathbf{45.1 \text{ kNm}}$$

Bending utilisation

$$M_{Ed} / M_{c,Rd} = \mathbf{59 \%}$$

**OK**

Design resistance for buckling

$$M_{b,Rd} = \mathbf{30.8 \text{ kNm}}$$

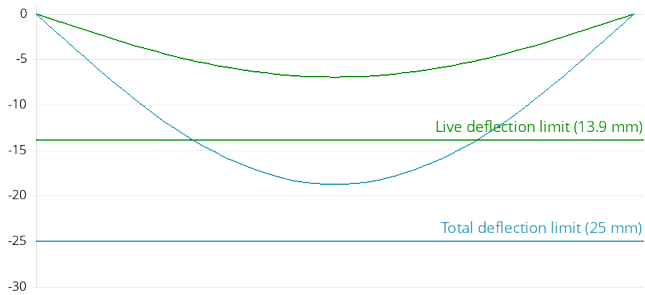
Buckling utilisation

$$M_{Ed} / M_{b,Rd} = \mathbf{86 \%}$$

**OK**

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## Deflection



Variable action deflection limit	$\Delta_Q = 13.9$ mm	
Variable action deflection	$\delta_Q = 7$ mm	<b>OK</b>
Total deflection limit	$\Delta_{G+Q} = 25$ mm	
Total deflection	$\delta_{G+Q} = 18.8$ mm	<b>OK</b>

## Section properties

Elastic modulus - major axis, yy	$W_{el} = 164$ cm <sup>3</sup>
Plastic modulus - major axis, yy	$W_{pl} = 182$ cm <sup>3</sup>
Second moment of area - major axis, yy	$I_y = 1,250$ cm <sup>4</sup>
Second moment of area - minor axis, zz	$I_z = 400$ cm <sup>4</sup>
Warping constant	$I_w = 0.021$ dm <sup>6</sup>
Torsional constant	$I_T = 4.63$ cm <sup>4</sup>
Area of section	$A = 2,920$ mm <sup>2</sup>

## 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 <sup>2</sup>	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 <sup>2</sup>	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	$\bar{\lambda}_{LT,0} = 0.4$	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

Nominal yield strength for S275 grade and nominal section thickness 6.80 mm	$f_y = 275$ N/mm <sup>2</sup>	Tata blue book
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## 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)

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Height of web	$h_w = 139$ mm	
Shear area for I and H sections	$A_v = 993$ mm <sup>2</sup>	cl.6.2.6 (3)
Design shear resistance	$V_{pl,Rd} = 158$ 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 (21 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	$W_{el} = 164,000$ mm <sup>3</sup>	
Design bending resistance	$M_{c,Rd} = 45.1$ 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 <sup>2</sup>	cl.3.2.6 (1)
Buckling length	$L_{cr} = 5,000$ mm	
Critical buckling moment	$M_{CR} = 42.7$ kNm	NCCI SN003b-EN-EU
Class 3 section, therefore use elastic modulus	$W_{el} = 164,000$ mm <sup>3</sup>	cl.6.3.2.1(3)
Non-dimensional slenderness ratio	$\bar{\lambda}_{LT} = 1.03$	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>b</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	$\phi_{LT} = 1$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = 0.683$	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	$\chi_{LT,mod} = 0.683$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = 30.8$ 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)