

Robinson Brown Design Unit 4 - Dalby Grove, Fairfield Park, Murton SR7 9GX 07812096701 info@robinsonbrowndesign.co.uk	Project 14 High Trees, South Shields, NE34 8EW	Project ref RBD/014HIGHTREES/002
	Calcs for Beam 2 - Over Existing Garage Door below New Extension	Date Nov 2024

Steel Beam Design

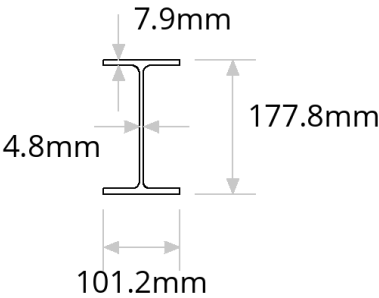
To Eurocode BS EN 1993-1-1/NA:2008

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	157 kN	20.2 kN	13 %	OK
Bending moment	47 kNm	12.8 kNm	27 %	OK
Buckling	35 kNm	12.8 kNm	37 %	OK
Total deflection	12.7 mm	2.2 mm	17 %	OK
Deflection due to variable actions	7 mm	0.5 mm	7 %	OK

Section details

Type	Universal beam
Section	178 x 102 x 19 UB
Steel grade	S275
Width	b = 101 mm
Depth	h = 178 mm
Web thickness	t _w = 4.8 mm
Flange thickness	t _f = 7.9 mm
Root radius	r = 7.6 mm
Mass per metre	w = 19 kg/m



Span and restraints

Effective span	L = 2,530 mm
Buckling length	L _{cr} = 2,530 mm

Deflection limits

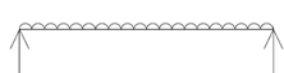
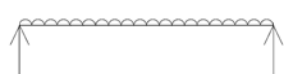
Variable action deflection limit	Δ _Q = L / 360 = 7.03 mm
Total deflection limit	Δ _{G+Q} = L / 200 = 12.6 mm

Safety factors

Partial factor for permanent actions	γ _G = 1.35
Partial factor for variable actions	γ _Q = 1.5

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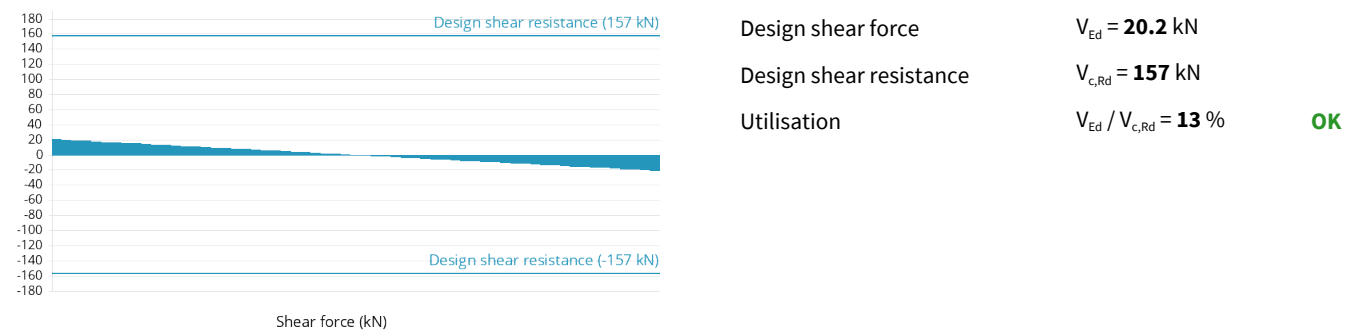
Loading details

	Self weight Permanent action	$SW = w \times 9.81 / 1000 = \mathbf{0.186 \text{ kN/m}}$
	Load 1: UDL - 102.5mm Brickwork Permanent action	$G_1 = 2 \text{ kN/m}^2 \times 3 \text{ m} = \mathbf{6 \text{ kN/m}}$
	Variable action	$Q_1 = 0 \text{ kN/m}^2 \times 3 \text{ m} = \mathbf{0 \text{ kN/m}}$
	Load 2: UDL - Sloping roof, 0° to 30° in North East England, Scotland, or 100m above sea level Permanent action	$G_2 = 1.15 \text{ kN/m}^2 \times 2.5 \text{ m} = \mathbf{2.88 \text{ kN/m}}$
	Variable action	$Q_2 = 1 \text{ kN/m}^2 \times 2.5 \text{ m} = \mathbf{2.5 \text{ kN/m}}$

Reactions

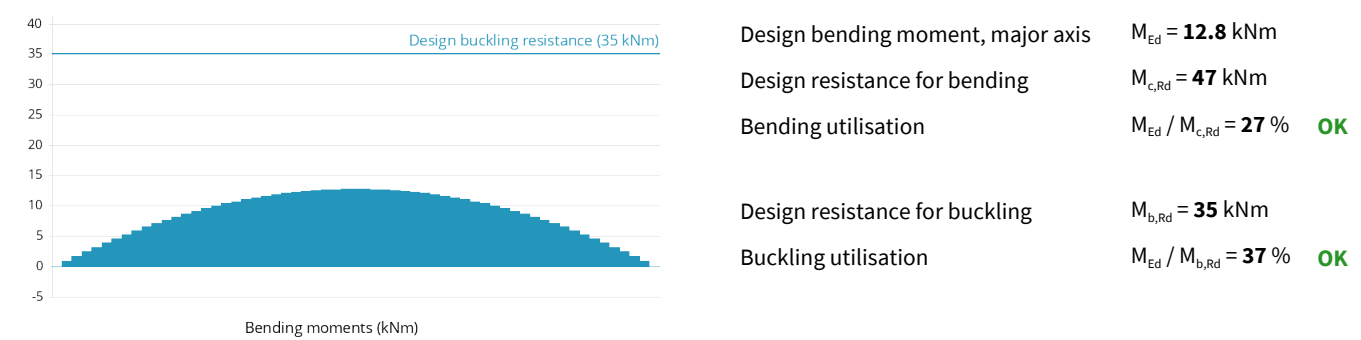
	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	11.5 kN	3.16 kN	14.6 kN	20.2 kN
Right reaction	11.5 kN	3.16 kN	14.6 kN	20.2 kN

Design shear force

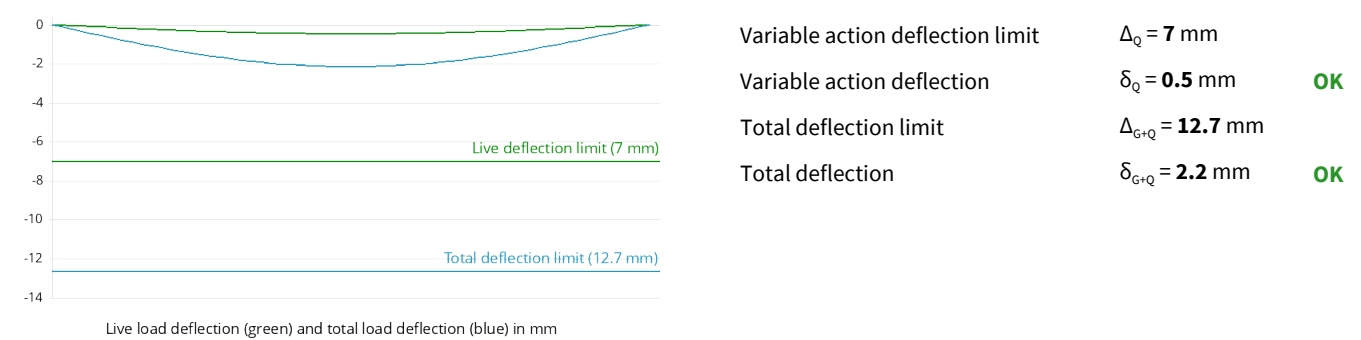


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Design bending moment



Deflection



Section properties

Elastic modulus - major axis, yy	$W_{el} = 153$ cm ³
Plastic modulus - major axis, yy	$W_{pl} = 171$ cm ³
Second moment of area - major axis, yy	$I_y = 1,360$ cm ⁴
Second moment of area - minor axis, zz	$I_z = 137$ cm ⁴
Warping constant	$I_w = 0.0099$ dm ⁶
Torsional constant	$I_T = 4.41$ cm ⁴
Area of section	$A = 2,430$ 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	$\bar{\lambda}_{LT,0} = 0.4$	cl.6.3.2.3(1) / BS-EN NA

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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 7.90 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 $c_f / t_f = 5.14$

Flange ratio limit for class 1 $9 \epsilon = 8.32$ Table 5.2 (sheet 2 of 3)

Flange class $\text{Class}_f = 1$

Web ratio for local buckling $c_w / t_w = 30.6$

Web ratio limit for class 1 $72 \epsilon = 66.6$ Table 5.2 (sheet 1 of 3)

Web class $\text{Class}_w = 1$

Section class $\text{Class} = 1$

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

Height of web $h_w = 162 \text{ mm}$

Shear area for I and H sections $A_v = 989 \text{ mm}^2$ cl.6.2.6 (3)

Design shear resistance $V_{pl,Rd} = 157 \text{ kN}$ 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 $h_w / t_w = 33.8$ 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 (20 kN) is less than half of the plastic shear resistance ($157 \text{ kN} / 2 = 79 \text{ kN}$), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus $W_{pl} = 171,000 \text{ mm}^3$

Design bending resistance $M_{c,Rd} = 47 \text{ kNm}$ eq (6.13)

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

C1 factor $C_1 = 1$

Shear modulus of elasticity $G_s = 81,000 \text{ N/mm}^2$ cl.3.2.6 (1)

Buckling length $L_{cr} = 2,530 \text{ mm}$

Critical buckling moment $M_{CR} = 54.8 \text{ kNm}$ NCCI SN003b-EN-EU

Class 1 section, therefore use plastic modulus $W_{pl} = 171,000 \text{ mm}^3$ cl.6.3.2.1(3)

Non-dimensional slenderness ratio $\bar{\lambda}_{LT} = 0.926$ 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 $\alpha_{LT} = 0.34$ Table 6.3 / BS-EN NA

Intermediate factor for reduction factor calculation $\phi_{LT} = 0.911$ cl.6.3.2.3 (1)

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Buckling reduction factor	$\chi_{LT} = \mathbf{0.744}$	eq (6.57)
Correction factor for moment distribution	$k_c = \mathbf{1}$	Table 6.6
Moment distribution modification factor	$f = \mathbf{1}$	cl.6.3.2.3 (2)
Modified buckling reduction factor	$\chi_{LT,mod} = \mathbf{0.744}$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = \mathbf{35\text{ kNm}}$	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)