

PlanningApplications.com Summer House, Upper Court Road, SURREY. CR3 7BF 0203 294 9477 www.planningapplications.com support@planningapplications.com	Project BEAM 1	Project ref 2023-7459
	Calcs for Connor McKitrich	Date 30 May 2023

Steel Beam Design

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

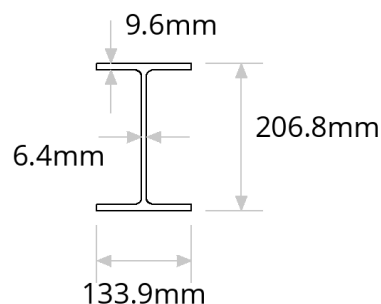
BEAM 1 - add 150mm bearing each end, total length 3880mm. Centre of connection point for BEAM 2 is 1224mm + 150mm = 1374mm from right.

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	$\Delta_Q = L / 360 = \mathbf{9.94 \text{ mm}}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = \mathbf{17.9 \text{ mm}}$

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Safety factors

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

Loading details



Self weight

Permanent action $SW = w \times 9.81 / 1000 = 0.294 \text{ kN/m}$



Load 1: UDL - Timber floor (domestic dwelling)

Permanent action $G_1 = 0.6 \text{ kN/m}^2 \times 1.5 \text{ m} = 0.9 \text{ kN/m}$

Variable action $Q_1 = 1.5 \text{ kN/m}^2 \times 1.5 \text{ m} = 2.25 \text{ kN/m}$



Load 2: UDL - Timber floor (domestic dwelling)

Permanent action $G_2 = 0.6 \text{ kN/m}^2 \times 1.5 \text{ m} = 0.9 \text{ kN/m}$

Variable action $Q_2 = 1.5 \text{ kN/m}^2 \times 1.5 \text{ m} = 2.25 \text{ kN/m}$



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

Permanent action $G_3 = 0 \text{ kN/m}^2 \times 3.58 \text{ m} = 0 \text{ kN/m}$

Variable action $Q_3 = 0.25 \text{ kN/m}^2 \times 3.58 \text{ m} = 0.895 \text{ kN/m}$



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

Permanent action $G_4 = 0 \text{ kN/m}^2 \times 5 \text{ m} = 0 \text{ kN/m}$

Variable action $Q_4 = 0.25 \text{ kN/m}^2 \times 5 \text{ m} = 1.25 \text{ kN/m}$



Load 5: UDL - Ceiling beneath sloping roof

Permanent action $G_5 = 0.3 \text{ kN/m}^2 \times 1.5 \text{ m} = 0.45 \text{ kN/m}$

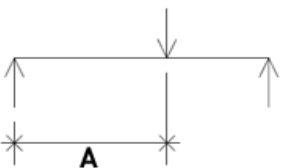
Variable action $Q_5 = 0.25 \text{ kN/m}^2 \times 1.5 \text{ m} = 0.375 \text{ kN/m}$



Load 6: UDL - Ceiling beneath sloping roof

Permanent action $G_6 = 0.3 \text{ kN/m}^2 \times 1.5 \text{ m} = 0.45 \text{ kN/m}$

Variable action $Q_6 = 0.25 \text{ kN/m}^2 \times 1.5 \text{ m} = 0.375 \text{ kN/m}$



Load 7: Point load - BEAM 2 Load

Distance to point load, A $A = 2.4 \text{ m}$

Permanent action $G_7 = 0.5 \text{ kN}$

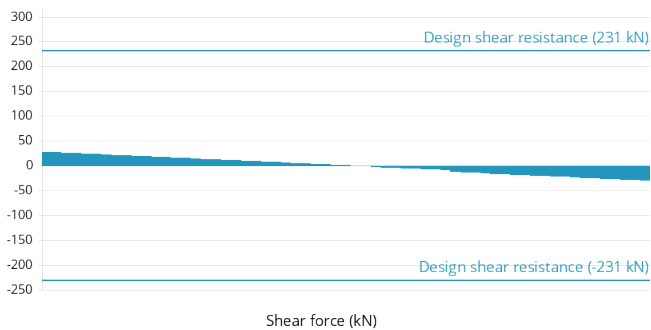
Variable action $Q_7 = 1.5 \text{ kN}$

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Reactions

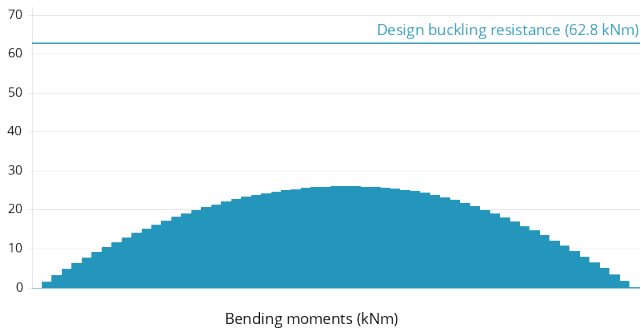
	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	5.52 kN	13.7 kN	19.3 kN	28.1 kN
Right reaction	5.69 kN	14.2 kN	19.9 kN	29.1 kN

Design shear force



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

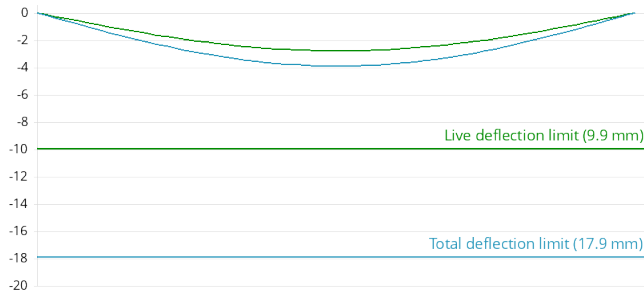
Design bending moment



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

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Deflection



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

Variable action deflection limit	$\Delta_Q = 9.9$ mm	
Variable action deflection	$\delta_Q = 2.8$ mm	OK
Total deflection limit	$\Delta_{G+Q} = 17.9$ mm	
Total deflection	$\delta_{G+Q} = 3.9$ 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	$\nu = 0.3$	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
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 9.60 mm	$f_y = 275$ N/mm ²	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	$c_f / t_f = 5.85$	
Flange ratio limit for class 1	$9 \epsilon = 8.32$	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	$72 \epsilon = 66.6$	Table 5.2 (sheet 1 of 3)

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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 $A_v = 1,460$ mm² 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 $(h_w / t_w) > (72 \epsilon / \eta)$

Web ratio for shear buckling $h_w / t_w = 29.3$ EN 1993-1-5:2006 cl.5.1 (2)

Shear buckling limit $72 \epsilon / \eta = 66.6$ 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 (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 $W_{pl} = 314,000$ mm³

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/mm² 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 $W_{pl} = 314,000$ mm³ 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 $\alpha_{LT} = 0.34$ Table 6.3 / BS-EN NA

Intermediate factor for reduction factor calculation $\phi_{LT} = 0.936$ cl.6.3.2.3 (1)

Buckling reduction factor $\chi_{LT} = 0.727$ 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.

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- 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)