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

9.3mm

203.2mm

5.4mm

101.8mm
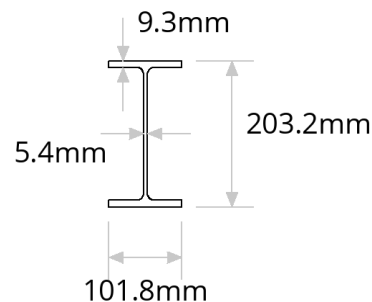
Span and restraints

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

Deflection limits

Variable action deflection limit	$\Delta_Q = L / 360 =$ **8.91** mm
Total deflection limit	$\Delta_{G+Q} = L / 200 =$ **16** mm

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	Calcs for **Connor McKitrich**	Date **30 May 2023**

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

$SW = w \times 9.81 / 1000 =$ **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
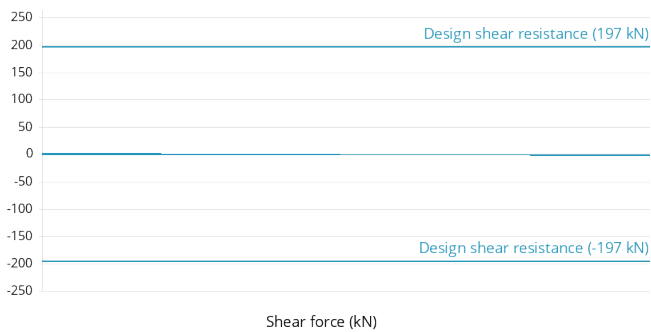
Shear force (kN)

Design shear force	V_{Ed} = **2.41** kN	
Design shear resistance	$V_{c,Rd}$ = **197** kN	
Utilisation	$V_{Ed} / V_{c,Rd}$ = **1** %	**OK**

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Design bending moment

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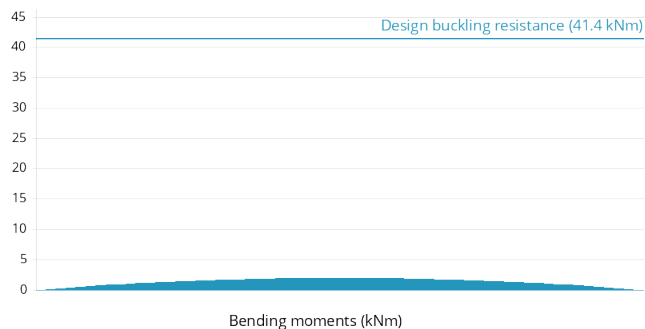
Design bending moment, major axis	$M_{Ed} = \mathbf{1.93}$ kNm
Design resistance for bending	$M_{c,Rd} = \mathbf{64.4}$ kNm
Bending utilisation	$M_{Ed} / M_{c,Rd} = \mathbf{3}$ % **OK**
Design resistance for buckling	$M_{b,Rd} = \mathbf{41.4}$ kNm
Buckling utilisation	$M_{Ed} / M_{b,Rd} = \mathbf{5}$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

Deflection

Live load deflection (green) and total load deflection (blue) in mm
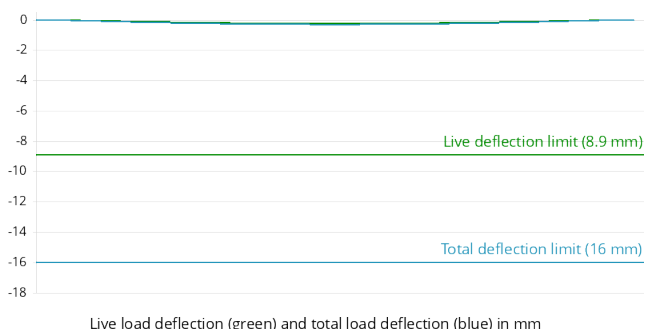
Variable action deflection limit	$\Delta_Q = \mathbf{8.9}$ mm
Variable action deflection	$\delta_Q = \mathbf{0.2}$ mm **OK**
Total deflection limit	$\Delta_{G+Q} = \mathbf{16}$ mm
Total deflection	$\delta_{G+Q} = \mathbf{0.3}$ mm **OK**

Section properties

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

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

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

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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

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Intermediate factor for reduction factor calculation	$\phi_{LT} = \mathbf{1.07}$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = \mathbf{0.643}$	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	$\chi_{LT,mod} = \mathbf{0.643}$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = \mathbf{41.4}$ 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)