



94 TITHEBARN ROAD, KNOWSLEY VILLAGE, KNOWSLEY, L34 0HA

**PROPOSED TWO-STOREY SIDE EXTENSION, SINGLE STOREY REAR EXTENSION
& ENTRANCE PORCH EXTENSION**

STRUCTURAL CALCULATIONS

Building Design Solutions 22 Sudworth Road, New Brighton, CH45 5BX 0151 630 1234 www.buildingdesignsolutions.com enquiries@buildingdesignsolutions.com	Project 94 Tithebarn Road, Knowsley Village, L34 0HA	Project ref
	Calcs for Beam No.1	Date 3 Nov 2021

Steel Beam Design

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

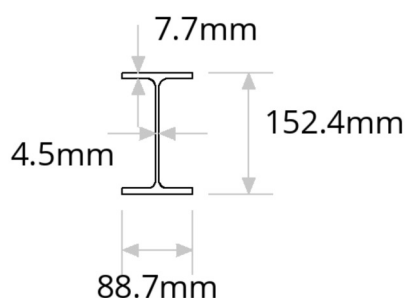
NOTE: Beam is to be sited within the existing floor zone, therefore fix continuous timber rails to both sides of the web and suspend the existing floor joists on galvanised joist hangers - to be secured to the timber rails.

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	130 kN	24.5 kN	19 %	OK
Bending moment	33.8 kNm	18.5 kNm	55 %	OK
Buckling	22.2 kNm	18.5 kNm	83 %	OK
Total deflection	14.6 mm	6.6 mm	45 %	OK
Deflection due to variable actions	8.1 mm	2.3 mm	29 %	OK

Section details

Type	Universal beam
Section	152 x 89 x 16 UB
Steel grade	S275
Width	b = 88.7 mm
Depth	h = 152 mm
Web thickness	t _w = 4.5 mm
Flange thickness	t _f = 7.7 mm
Root radius	r = 7.6 mm
Mass per metre	w = 16 kg/m



Span and restraints

Effective span	L = 2,920 mm
Buckling length	L _{cr} = 2,920 mm

Deflection limits

Variable action deflection limit	$\Delta_Q = L / 360 = \mathbf{8.13 \text{ mm}}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = \mathbf{14.6 \text{ mm}}$

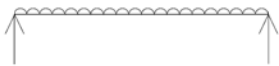
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Safety factors

Partial factor for permanent actions $\gamma_G = 1.35$

Partial factor for variable actions $\gamma_Q = 1.5$

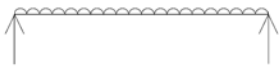
Loading details



Self weight

Permanent action

$$SW = w \times 9.81 / 1000 = \mathbf{0.157 \text{ kN/m}}$$



Load 1: UDL - Timber floor (domestic dwelling)

Permanent action

$$G_1 = 0.6 \text{ kN/m}^2 \times 2.85 \text{ m} = \mathbf{1.71 \text{ kN/m}}$$

Variable action

$$Q_1 = 1.5 \text{ kN/m}^2 \times 2.85 \text{ m} = \mathbf{4.28 \text{ kN/m}}$$



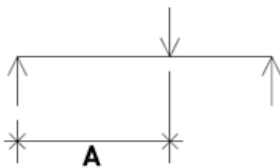
Load 2: UDL - Masonry Wall

Permanent action

$$G_2 = 1.5 \text{ kN/m}^2 \times 2.4 \text{ m} = \mathbf{3.6 \text{ kN/m}}$$

Variable action

$$Q_2 = 0 \text{ kN/m}^2 \times 2.4 \text{ m} = \mathbf{0 \text{ kN/m}}$$



Load 3: Point load - PL from partition

Distance to point load, A

$$A = \mathbf{1.05 \text{ m}}$$

Permanent action

$$G_3 = \mathbf{5 \text{ kN}}$$

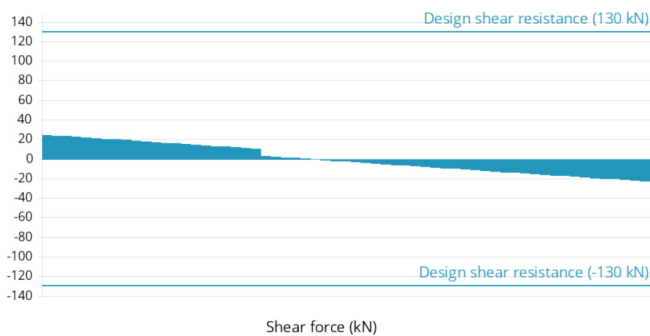
Variable action

$$Q_3 = \mathbf{0 \text{ kN}}$$

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	11.2 kN	6.25 kN	17.5 kN	24.5 kN
Right reaction	9.79 kN	6.25 kN	16 kN	22.6 kN

Design shear force



Design shear force

$$V_{Ed} = \mathbf{24.5 \text{ kN}}$$

Design shear resistance

$$V_{c,Rd} = \mathbf{130 \text{ kN}}$$

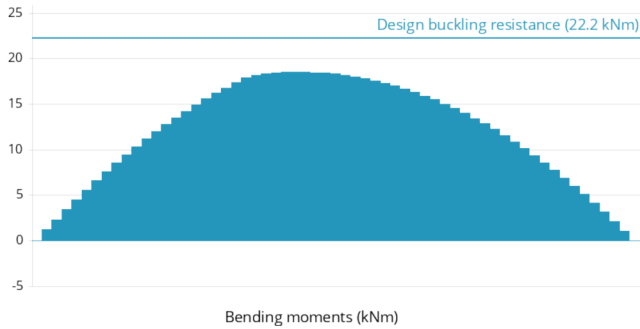
Utilisation

$$V_{Ed} / V_{c,Rd} = \mathbf{19 \%}$$

OK

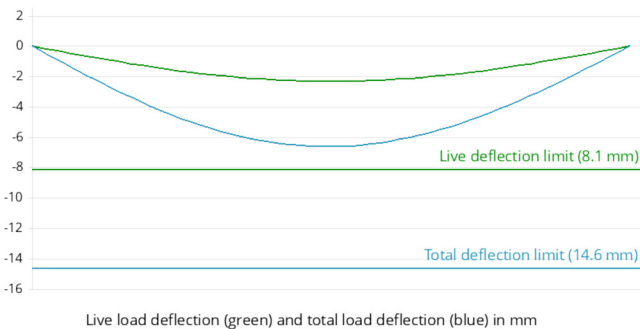
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Design bending moment



Design bending moment, major axis	$M_{Ed} = 18.5 \text{ kNm}$
Design resistance for bending	$M_{c,Rd} = 33.8 \text{ kNm}$
Bending utilisation	$M_{Ed} / M_{c,Rd} = 55 \% \text{ OK}$
Design resistance for buckling	$M_{b,Rd} = 22.2 \text{ kNm}$
Buckling utilisation	$M_{Ed} / M_{b,Rd} = 83 \% \text{ OK}$

Deflection



Variable action deflection limit	$\Delta_Q = 8.1 \text{ mm}$
Variable action deflection	$\delta_Q = 2.3 \text{ mm} \text{ OK}$
Total deflection limit	$\Delta_{G+Q} = 14.6 \text{ mm}$
Total deflection	$\delta_{G+Q} = 6.6 \text{ mm} \text{ OK}$

Section properties

Elastic modulus - major axis, yy	$W_{el} = 109 \text{ cm}^3$
Plastic modulus - major axis, yy	$W_{pl} = 123 \text{ cm}^3$
Second moment of area - major axis, yy	$I_y = 834 \text{ cm}^4$
Second moment of area - minor axis, zz	$I_z = 89.8 \text{ cm}^4$
Warping constant	$I_w = 0.0047 \text{ dm}^6$
Torsional constant	$I_t = 3.56 \text{ cm}^4$
Area of section	$A = 2,030 \text{ mm}^2$

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

Young's modulus of elasticity	$E = 210,000 \text{ N/mm}^2$	cl.3.2.6
Poisson's ratio in elastic stage	$\nu = 0.3$	cl.3.2.6
Shear modulus	$G_s = 81,000 \text{ N/mm}^2$	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 7.70 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

$$c_f / t_f = 4.48$$

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

$$c_w / t_w = 27.1$$

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

$$h_w = 137 \text{ mm}$$

Shear area for I and H sections

$$A_v = 816 \text{ mm}^2$$

cl.6.2.6 (3)

Design shear resistance

$$V_{pl,Rd} = 130 \text{ kN}$$

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

$$h_w / t_w = 30.4$$

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 (24 kN) is less than half of the plastic shear resistance ($130 \text{ kN} / 2 = 65 \text{ kN}$), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus

$$W_{pl} = 123,000 \text{ mm}^3$$

Design bending resistance

$$M_{c,Rd} = 33.8 \text{ kNm}$$

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

$$L_{cr} = 2,920 \text{ mm}$$

Critical buckling moment

$$M_{CR} = 29.6 \text{ kNm}$$

NCCI SN003b-EN-EU

Class 1 section, therefore use plastic modulus

$$W_{pl} = 123,000 \text{ mm}^3$$

cl.6.3.2.1(3)

Non-dimensional slenderness ratio

$$\bar{\lambda}_{LT} = 1.07$$

cl.6.3.2.2 (1)

Depth to width ratio for buckling curve

$$h / b = 1.72$$

Buckling curve for h / b ratio

$$\text{Buckling curve} = b$$

Table 6.5 / BS-EN NA

Imperfection factor for buckling curve b

$$\alpha_{LT} = 0.34$$

Table 6.3 / BS-EN NA

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Intermediate factor for reduction factor calculation	$\phi_{LT} = \mathbf{1.04}$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = \mathbf{0.657}$	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	$\chi_{LT,mod} = \mathbf{0.657}$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = \mathbf{22.2}$ 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;

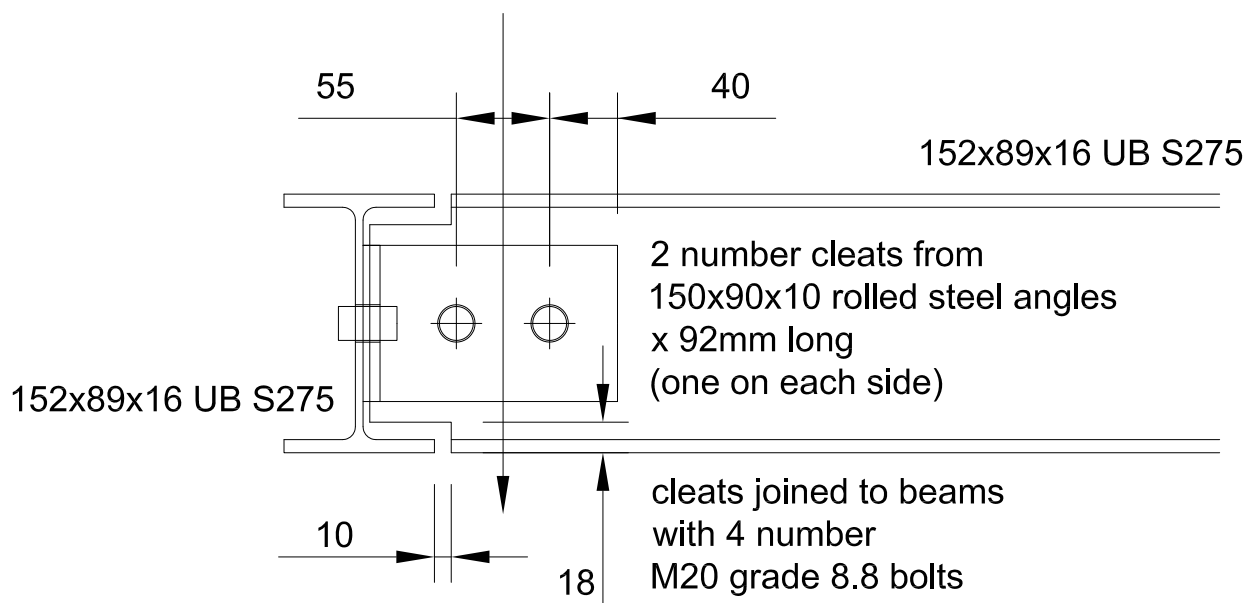
- 1) End of beam built into masonry wall.
- 2) End of beam fixed to a masonry wall.
- 3) End of beam fixed to a column or a beam.

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)

Connection Detail 152x89x16 UB

maximum factored load = 20 kN



Bearing Capacity of M20 bolt passing through 4.5mm thick
S275 steel = 37.9 kN

$2 \times 37.9 \text{ kN} = 75.8 \text{ kN} > 20 \text{ kN}$ therefore OK

Shearing Resistance of M20 grade 8.8 bolt = 94.1 kN

$2 \times 94.1 \text{ kN} = 188.2 \text{ kN} > 20 \text{ kN}$ therefore OK

Values from P363 Steel Building Design: Design Data, In
accordance with Eurocodes and the UK National Annexes.

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	Calcs for Beam No.2	Date 3 Nov 2021

Steel Beam Design

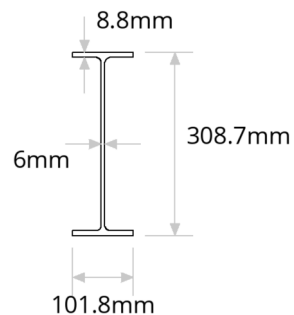
To Eurocode BS EN 1993-1-1/NA:2008

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	315 kN	36.3 kN	12 %	OK
Bending moment	111 kNm	59.2 kNm	53 %	OK
Buckling	111 kNm	59.2 kNm	53 %	OK
Total deflection	26.2 mm	9.9 mm	38 %	OK
Deflection due to variable actions	14.5 mm	1.9 mm	13 %	OK

Section details

Type	Universal beam
Section	305 x 102 x 28 UB
Steel grade	S275
Width	b = 102 mm
Depth	h = 309 mm
Web thickness	t _w = 6 mm
Flange thickness	t _f = 8.8 mm
Root radius	r = 7.6 mm
Mass per metre	w = 28.2 kg/m



Span and restraints

Effective span	L = 5,230 mm
Buckling length	L _{cr} = 750 mm

Deflection limits

Variable action deflection limit	$\Delta_Q = L / 360 = \mathbf{14.5 \text{ mm}}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = \mathbf{26.2 \text{ mm}}$

Safety factors

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

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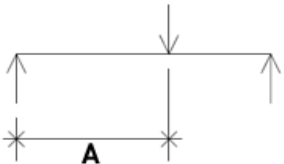
Loading details



Self weight

Permanent action

$$SW = w \times 9.81 / 1000 = \mathbf{0.277 \text{ kN/m}}$$



Load 1: Point load - PL from Beam No.1

Distance to point load, A

$$A = \mathbf{2.92 \text{ m}}$$

Permanent action

$$G_1 = \mathbf{9.79 \text{ kN}}$$

Variable action

$$Q_1 = \mathbf{6.25 \text{ kN}}$$



Load 2: UDL - 102.5mm Brickwork + Plaster or render on ONE side

Permanent action

$$G_2 = \mathbf{2.25 \text{ kN/m}^2} \times \mathbf{2.4 \text{ m}} = \mathbf{5.4 \text{ kN/m}}$$

Variable action

$$Q_2 = \mathbf{0 \text{ kN/m}^2} \times \mathbf{2.4 \text{ m}} = \mathbf{0 \text{ kN/m}}$$



Load 3: UDL - Sloping roof, 30° to 45°

Permanent action

$$G_3 = \mathbf{1.41 \text{ kN/m}^2} \times \mathbf{0.4 \text{ m}} = \mathbf{0.564 \text{ kN/m}}$$

Variable action

$$Q_3 = \mathbf{0.75 \text{ kN/m}^2} \times \mathbf{0.4 \text{ m}} = \mathbf{0.3 \text{ kN/m}}$$



Load 4: UDL - Ceiling beneath sloping roof

Permanent action

$$G_4 = \mathbf{0.3 \text{ kN/m}^2} \times \mathbf{0.2 \text{ m}} = \mathbf{0.06 \text{ kN/m}}$$

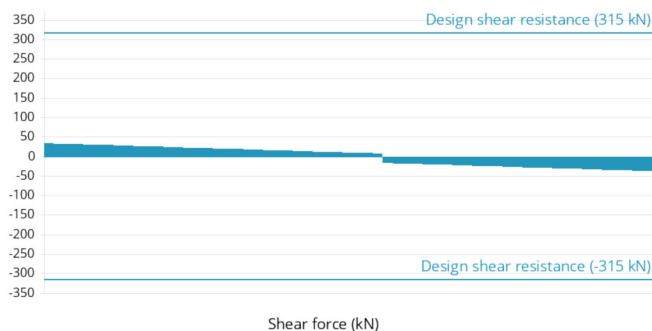
Variable action

$$Q_4 = \mathbf{0.25 \text{ kN/m}^2} \times \mathbf{0.2 \text{ m}} = \mathbf{0.05 \text{ kN/m}}$$

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	20.8 kN	3.67 kN	24.5 kN	33.6 kN
Right reaction	22 kN	4.41 kN	26.4 kN	36.3 kN

Design shear force



Design shear force

$$V_{Ed} = \mathbf{36.3 \text{ kN}}$$

Design shear resistance

$$V_{c,Rd} = \mathbf{315 \text{ kN}}$$

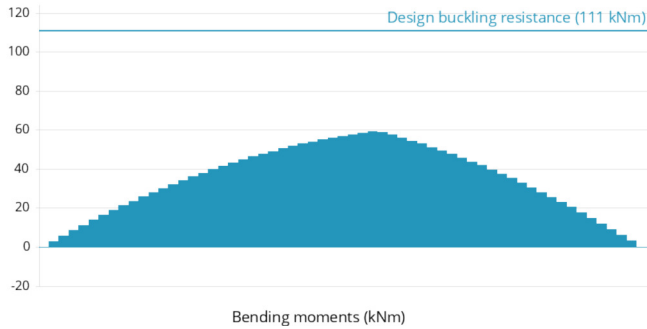
Utilisation

$$V_{Ed} / V_{c,Rd} = \mathbf{12 \%}$$

OK

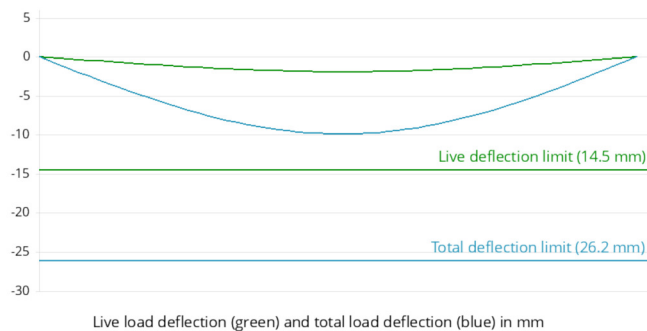
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Design bending moment



Design bending moment, major axis	$M_{Ed} = 59.2$ kNm
Design resistance for bending	$M_{c,Rd} = 111$ kNm
Bending utilisation	$M_{Ed} / M_{c,Rd} = 53\%$ OK
Design resistance for buckling	$M_{b,Rd} = 111$ kNm
Buckling utilisation	$M_{Ed} / M_{b,Rd} = 53\%$ OK

Deflection



Variable action deflection limit	$\Delta_Q = 14.5$ mm
Variable action deflection	$\delta_Q = 1.9$ mm OK
Total deflection limit	$\Delta_{G+Q} = 26.2$ mm
Total deflection	$\delta_{G+Q} = 9.9$ mm OK

Section properties

Elastic modulus - major axis, yy	$W_{el} = 348$ cm ³
Plastic modulus - major axis, yy	$W_{pl} = 403$ cm ³
Second moment of area - major axis, yy	$I_y = 5,370$ cm ⁴
Second moment of area - minor axis, zz	$I_z = 155$ cm ⁴
Warping constant	$I_w = 0.0349$ dm ⁶
Torsional constant	$I_t = 7.4$ cm ⁴
Area of section	$A = 3,590$ 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 8.80 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

$$c_f / t_f = 4.58$$

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

$$c_w / t_w = 46$$

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

$$h_w = 291 \text{ mm}$$

Shear area for I and H sections

$$A_v = 1,980 \text{ mm}^2$$

cl.6.2.6 (3)

Design shear resistance

$$V_{pl,Rd} = 315 \text{ kN}$$

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

$$h_w / t_w = 48.5$$

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 (36 kN) is less than half of the plastic shear resistance (315 kN / 2 = 158 kN), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus

$$W_{pl} = 403,000 \text{ mm}^3$$

Design bending resistance

$$M_{c,Rd} = 111 \text{ kNm}$$

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

$$L_{cr} = 750 \text{ mm}$$

Critical buckling moment

$$M_{CR} = 877 \text{ kNm}$$

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Class 1 section, therefore use plastic modulus

$$W_{pl} = 403,000 \text{ mm}^3$$

cl.6.3.2.1(3)

Non-dimensional slenderness ratio

$$\bar{\lambda}_{LT} = 0.356$$

cl.6.3.2.2 (1)

Depth to width ratio for buckling curve

$$h / b = 3.03$$

Buckling curve for h / b ratio

$$\text{Buckling curve} = c$$

Table 6.5 / BS-EN NA

Imperfection factor for buckling curve c

$$\alpha_{LT} = 0.49$$

Table 6.3 / BS-EN NA

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Intermediate factor for reduction factor calculation	$\phi_{LT} = \mathbf{0.537}$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = \mathbf{1}$	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	$\chi_{LT,mod} = \mathbf{1}$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = \mathbf{111}$ 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)

Beam No.2 Padstones Calculation

COMPLIES WITH LATEST EUROPEAN DESIGN CODES

NOTE: Internal leaf of cavity wall assumed to be 100mm dense blockwork.

Beam End Reaction = **36.30** kN (factored) Variable Load Safety Factor = 1.5
Factored Load at End of Beam Permanent Load Safety Factor = 1.35

Characteristic strength of masonry = **4.2** N/mm² (Brickwork usually = 4.5 N/mm²)
(3.6N Blockwork usually = 2.6 N/mm²)
Width of beam end bearing = **102** mm (A Engineering Brick = 13.2 N/mm²)
Length of beam end bearing = **150** mm (B Engineering Brick = 10.5 N/mm²)
(Weak Brickwork = approx 2.8 N/mm²)
(7.3N Blockwork usually = 4.2 N/mm²)
(10.4N Blockwork usually = 5.4 N/mm²)

$\gamma_m = 3.0$

Bearing Factor = **1.25**

Results

Maximum Bearing Stress = 1.75 N/mm²
Actual Bearing Stress = 2.37 N/mm²

Padstone Required

Padstone Results

Characteristic strength of Padstone = **10.5** N/mm² (A Engineering Brick = 13.2 N/mm²)
(B Engineering Brick = 10.5 N/mm²)
Width of Padstone = **100** mm (Concrete C15 = 15 N/mm²)
Length of Padstone = **215** mm (Concrete C30 = 30 N/mm²)
(Concrete C40 = 40 N/mm²)
(Steel Plate = 275 N/mm²)

Allowable padstone stress = 4.38 N/mm²
Stress under beam end bearing = 2.37 N/mm² Therefore Padstone Stress OK
Allowable masonry stress = 1.75 N/mm²
Stress under padstone = 1.69 N/mm² Therefore Masonry Stress OK

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Steel Beam Design

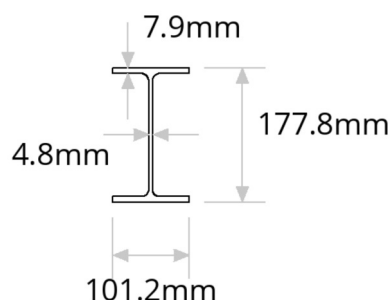
To Eurocode BS EN 1993-1-1/NA:2008

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	157 kN	7.97 kN	5 %	OK
Bending moment	47 kNm	7.97 kNm	17 %	OK
Buckling	25.5 kNm	7.97 kNm	31 %	OK
Total deflection	20 mm	3.3 mm	16 %	OK
Deflection due to variable actions	11.1 mm	1.6 mm	15 %	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 = 4,000 mm
Buckling length	L _{cr} = 4,000 mm

Deflection limits

Variable action deflection limit	$\Delta_Q = L / 360 = \mathbf{11.1 \text{ mm}}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = \mathbf{20 \text{ mm}}$

Safety factors

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

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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 - Flat roof, with no permanent access

Permanent action

$$G_1 = 1 \text{ kN/m}^2 \times 0.6 \text{ m} = \mathbf{0.6 \text{ kN/m}}$$

Variable action

$$Q_1 = 0.75 \text{ kN/m}^2 \times 0.6 \text{ m} = \mathbf{0.45 \text{ kN/m}}$$



Load 2: UDL - Glazed roof lantern

Permanent action

$$G_2 = 0.5 \text{ kN/m}^2 \times 1.25 \text{ m} = \mathbf{0.625 \text{ kN/m}}$$

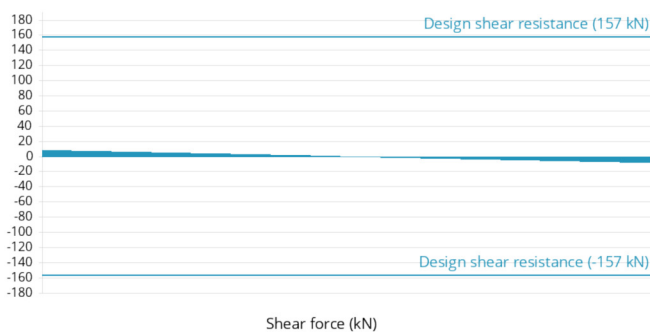
Variable action

$$Q_2 = 0.75 \text{ kN/m}^2 \times 1.25 \text{ m} = \mathbf{0.938 \text{ kN/m}}$$

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	2.82 kN	2.78 kN	5.6 kN	7.97 kN
Right reaction	2.82 kN	2.78 kN	5.6 kN	7.97 kN

Design shear force



Design shear force

$$V_{Ed} = \mathbf{7.97 \text{ kN}}$$

Design shear resistance

$$V_{c,Rd} = \mathbf{157 \text{ kN}}$$

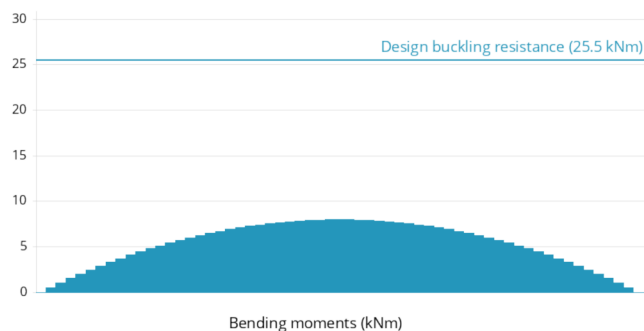
Utilisation

$$V_{Ed} / V_{c,Rd} = \mathbf{5 \%}$$

OK

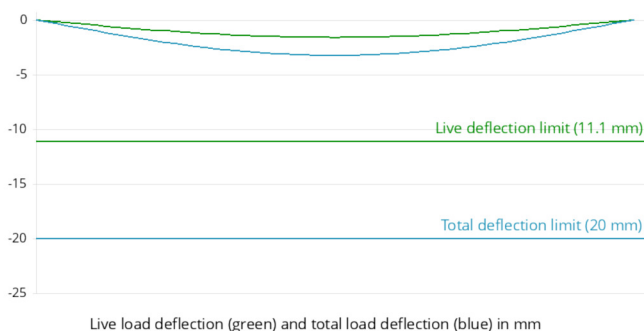
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Design bending moment



Design bending moment, major axis	$M_{Ed} = 7.97 \text{ kNm}$
Design resistance for bending	$M_{c,Rd} = 47 \text{ kNm}$
Bending utilisation	$M_{Ed} / M_{c,Rd} = 17 \% \text{ OK}$
Design resistance for buckling	$M_{b,Rd} = 25.5 \text{ kNm}$
Buckling utilisation	$M_{Ed} / M_{b,Rd} = 31 \% \text{ OK}$

Deflection



Variable action deflection limit	$\Delta_Q = 11.1 \text{ mm}$
Variable action deflection	$\delta_Q = 1.6 \text{ mm} \text{ OK}$
Total deflection limit	$\Delta_{G+Q} = 20 \text{ mm}$
Total deflection	$\delta_{G+Q} = 3.3 \text{ mm} \text{ OK}$

Section properties

Elastic modulus - major axis, yy	$W_{el} = 153 \text{ cm}^3$
Plastic modulus - major axis, yy	$W_{pl} = 171 \text{ cm}^3$
Second moment of area - major axis, yy	$I_y = 1,360 \text{ cm}^4$
Second moment of area - minor axis, zz	$I_z = 137 \text{ cm}^4$
Warping constant	$I_w = 0.0099 \text{ dm}^6$
Torsional constant	$I_T = 4.41 \text{ cm}^4$
Area of section	$A = 2,430 \text{ mm}^2$

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

Young's modulus of elasticity	$E = 210,000 \text{ N/mm}^2$	cl.3.2.6
Poisson's ratio in elastic stage	$\nu = 0.3$	cl.3.2.6
Shear modulus	$G_s = 81,000 \text{ N/mm}^2$	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	$\bar{\lambda}_{L,T,0} = 0.4$	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 (8 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 $L_{cr} = 4,000 \text{ mm}$

Critical buckling moment $M_{CR} = 29.4 \text{ kNm}$ 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 $\bar{\lambda}_{LT} = 1.27$ 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

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Intermediate factor for reduction factor calculation	$\phi_{LT} = 1.25$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = 0.542$	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	$\chi_{LT,mod} = 0.542$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = 25.5 \text{ kNm}$	eq (6.55)

Notes

C1 value conservatively taken as 1.0

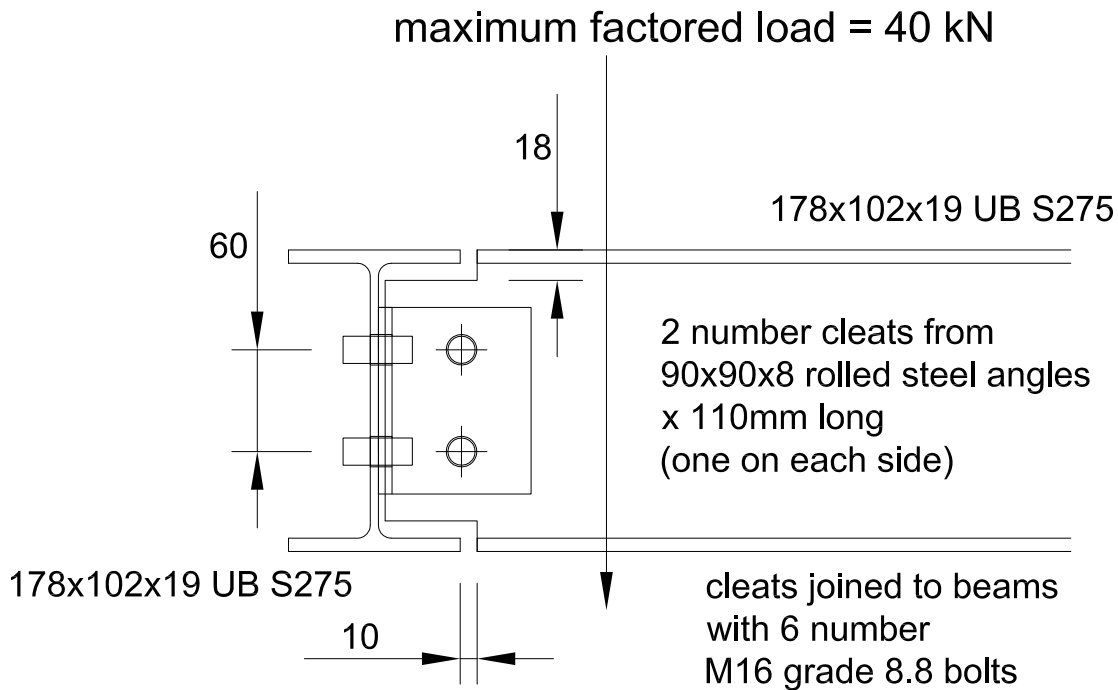
Ends of beam are to be laterally restrained. Ends of beams can be laterally restrained using one of the following methods;

- 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)

Connection Detail 178x102x19 UB



Bearing Capacity of M16 bolt passing through 4.8mm thick
S275 steel = 32.7 kN

$2 \times 32.7 \text{ kN} = 65.4 \text{ kN} > 40 \text{ kN}$ therefore OK

Shearing Resistance of M16 grade 8.8 bolt = 188 kN

$2 \times 188 \text{ kN} = 376 \text{ kN} > 40 \text{ kN}$ therefore OK

Values from P363 Steel Building Design: Design Data, In
accordance with Eurocodes and the UK National Annexes.

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Steel Beam Design

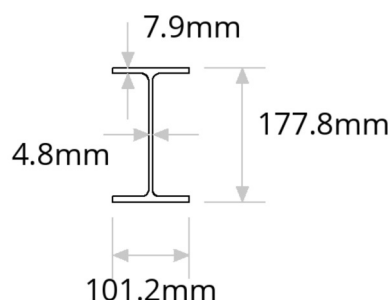
To Eurocode BS EN 1993-1-1/NA:2008

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	157 kN	12.6 kN	8 %	OK
Bending moment	47 kNm	14.2 kNm	30 %	OK
Buckling	23.9 kNm	14.2 kNm	59 %	OK
Total deflection	21.6 mm	7.2 mm	33 %	OK
Deflection due to variable actions	12 mm	3.3 mm	27 %	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 = 4,320 mm
Buckling length	L _{cr} = 4,320 mm

Deflection limits

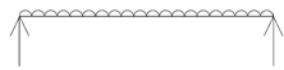
Variable action deflection limit	$\Delta_Q = L / 360 = \mathbf{12 \text{ mm}}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = \mathbf{21.6 \text{ mm}}$

Safety factors

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

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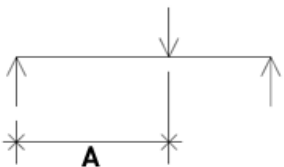
Loading details



Self weight

Permanent action

$$SW = w \times 9.81 / 1000 = \mathbf{0.186 \text{ kN/m}}$$



Load 1: Point load - PL from Beam No.3

Distance to point load, A

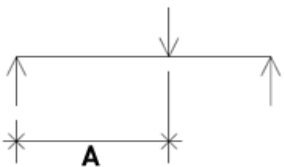
$$A = \mathbf{1.15 \text{ m}}$$

Permanent action

$$G_1 = \mathbf{2.82 \text{ kN}}$$

Variable action

$$Q_1 = \mathbf{2.78 \text{ kN}}$$



Load 2: Point load - PL from Beam No.3

Distance to point load, A

$$A = \mathbf{3.15 \text{ m}}$$

Permanent action

$$G_2 = \mathbf{2.82 \text{ kN}}$$

Variable action

$$Q_2 = \mathbf{2.78 \text{ kN}}$$



Load 3: UDL - Flat roof, with no permanent access

Permanent action

$$G_3 = \mathbf{1 \text{ kN/m}^2} \times \mathbf{0.75 \text{ m}} = \mathbf{0.75 \text{ kN/m}}$$

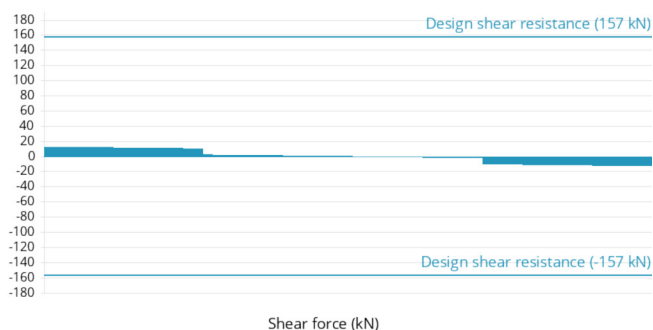
Variable action

$$Q_3 = \mathbf{0.75 \text{ kN/m}^2} \times \mathbf{0.75 \text{ m}} = \mathbf{0.562 \text{ kN/m}}$$

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	4.86 kN	4.01 kN	8.86 kN	12.6 kN
Right reaction	4.83 kN	3.98 kN	8.81 kN	12.5 kN

Design shear force



Design shear force

$$V_{Ed} = \mathbf{12.6 \text{ kN}}$$

Design shear resistance

$$V_{c,Rd} = \mathbf{157 \text{ kN}}$$

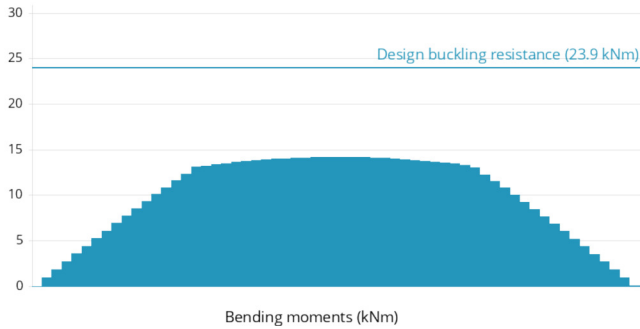
Utilisation

$$V_{Ed} / V_{c,Rd} = \mathbf{8 \%}$$

OK

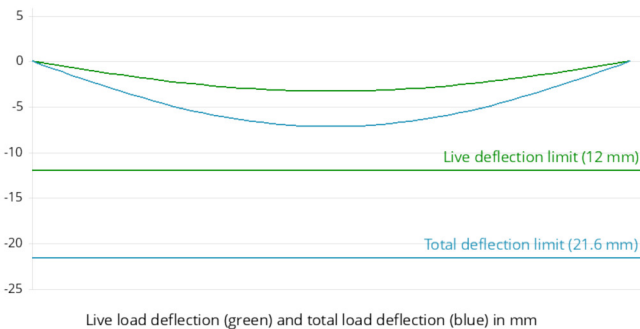
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Design bending moment



Design bending moment, major axis	$M_{Ed} = 14.2$ kNm
Design resistance for bending	$M_{c,Rd} = 47$ kNm
Bending utilisation	$M_{Ed} / M_{c,Rd} = 30$ % OK
Design resistance for buckling	$M_{b,Rd} = 23.9$ kNm
Buckling utilisation	$M_{Ed} / M_{b,Rd} = 59$ % OK

Deflection



Variable action deflection limit	$\Delta_Q = 12$ mm
Variable action deflection	$\delta_Q = 3.3$ mm OK
Total deflection limit	$\Delta_{G+Q} = 21.6$ mm
Total deflection	$\delta_{G+Q} = 7.2$ mm OK

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 (13 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 $L_{cr} = 4,320 \text{ mm}$

Critical buckling moment $M_{cr} = 26.7 \text{ kNm}$ 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 $\bar{\lambda}_{LT} = 1.33$ 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

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Intermediate factor for reduction factor calculation	$\phi_{LT} = \mathbf{1.32}$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = \mathbf{0.509}$	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	$\chi_{LT,mod} = \mathbf{0.509}$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = \mathbf{23.9}$ 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)

Beam No.4: Padstone on Existing Rear Wall Calculation

COMPLIES WITH LATEST EUROPEAN DESIGN CODES

Beam End Reaction = **12.60** kN (factored) Variable Load Safety Factor = 1.5
Factored Load at End of Beam Permanent Load Safety Factor = 1.35

Characteristic strength of masonry = **4.5** N/mm² (Brickwork usually = 4.5 N/mm²)
(3.6N Blockwork usually = 2.6 N/mm²)
Width of beam end bearing = **102** mm (A Engineering Brick = 13.2 N/mm²)
Length of beam end bearing = **100** mm (B Engineering Brick = 10.5 N/mm²)
(Weak Brickwork = approx 2.8 N/mm²)
(7.3N Blockwork usually = 4.2 N/mm²)
(10.4N Blockwork usually = 5.4 N/mm²)

$\gamma_m = 3.0$

Bearing Factor = **1.25**

Results

Maximum Bearing Stress = 1.88 N/mm²
Actual Bearing Stress = 1.24 N/mm²

Padstone Not Required

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Steel Beam Design

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

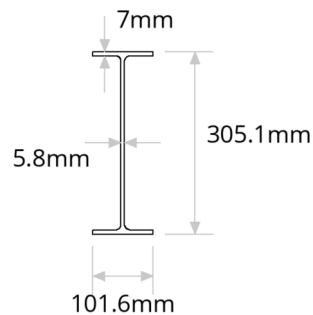
NOTE: Beam No.5 and Beam No.2 are to be bolted together at 750mm centres

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	299 kN	31.6 kN	11 %	OK
Bending moment	94 kNm	41.6 kNm	44 %	OK
Buckling	94 kNm	41.6 kNm	44 %	OK
Total deflection	26.2 mm	9.2 mm	35 %	OK
Deflection due to variable actions	14.5 mm	1.6 mm	11 %	OK

Section details

Type	Universal beam
Section	305 x 102 x 25 UB
Steel grade	S275
Width	b = 102 mm
Depth	h = 305 mm
Web thickness	t _w = 5.8 mm
Flange thickness	t _f = 7 mm
Root radius	r = 7.6 mm
Mass per metre	w = 24.8 kg/m



Span and restraints

Effective span	L = 5,230 mm
Buckling length	L _{cr} = 750 mm

Deflection limits

Variable action deflection limit	$\Delta_Q = L / 360 = \mathbf{14.5 \text{ mm}}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = \mathbf{26.2 \text{ mm}}$

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	Calcs for Beam No.5	Date 14 May 2022

Safety factors

Partial factor for permanent actions $\gamma_G = 1.35$

Partial factor for variable actions $\gamma_Q = 1.5$

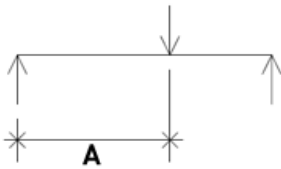
Loading details



Self weight

Permanent action

$$SW = w \times 9.81 / 1000 = \mathbf{0.243 \text{ kN/m}}$$



Load 1: Point load - PL from Beam No.4

Distance to point load, A

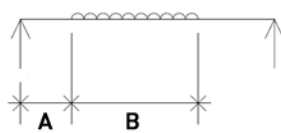
$$A = \mathbf{1.54 \text{ m}}$$

Permanent action

$$G_1 = \mathbf{4.86 \text{ kN}}$$

Variable action

$$Q_1 = \mathbf{4.83 \text{ kN}}$$



Load 2: Partial UDL - Flat roof, with no permanent access

Dimension to start of load, A

$$A = \mathbf{1.54 \text{ m}}$$

Length of load, B

$$B = \mathbf{3.69 \text{ m}}$$

Permanent action

$$G_2 = 1 \text{ kN/m}^2 \times 0.6 \text{ m} = \mathbf{0.6 \text{ kN/m}}$$

Variable action

$$Q_2 = 0.75 \text{ kN/m}^2 \times 0.6 \text{ m} = \mathbf{0.45 \text{ kN/m}}$$



Load 3: UDL - 102.5mm Brickwork + Plaster or render on ONE side

Permanent action

$$G_3 = 2.25 \text{ kN/m}^2 \times 2.4 \text{ m} = \mathbf{5.4 \text{ kN/m}}$$

Variable action

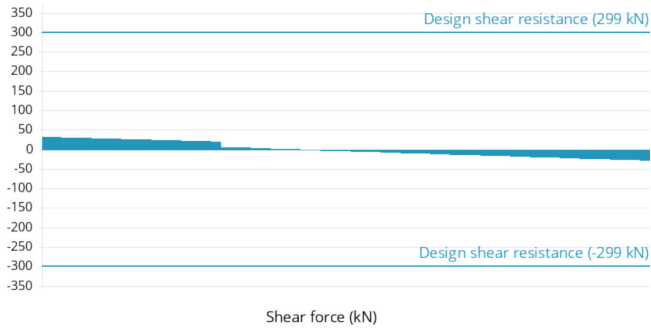
$$Q_3 = 0 \text{ kN/m}^2 \times 2.4 \text{ m} = \mathbf{0 \text{ kN/m}}$$

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	19 kN	3.99 kN	23 kN	31.6 kN
Right reaction	17.6 kN	2.5 kN	20.1 kN	27.5 kN

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Design shear force



Design shear force

$$V_{Ed} = 31.6 \text{ kN}$$

Design shear resistance

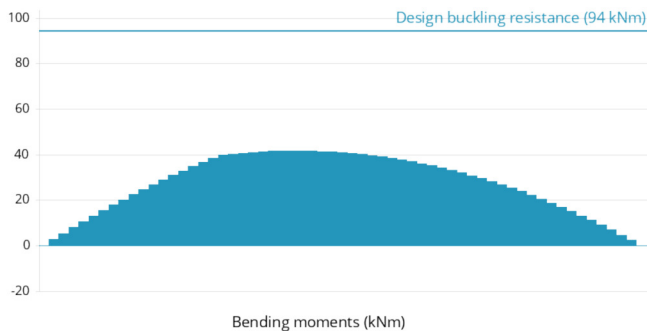
$$V_{c,Rd} = 299 \text{ kN}$$

Utilisation

$$V_{Ed} / V_{c,Rd} = 11 \%$$

OK

Design bending moment



Design bending moment, major axis

$$M_{Ed} = 41.6 \text{ kNm}$$

Design resistance for bending

$$M_{c,Rd} = 94 \text{ kNm}$$

Bending utilisation

$$M_{Ed} / M_{c,Rd} = 44 \%$$

OK

Design resistance for buckling

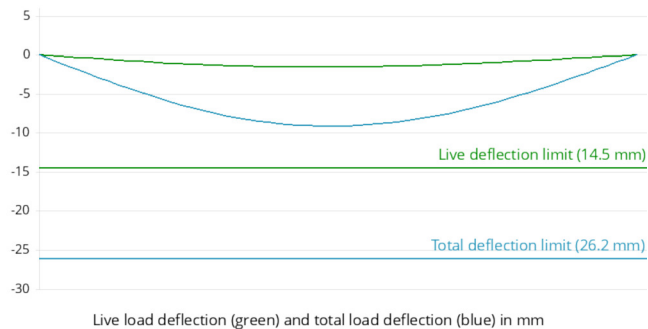
$$M_{b,Rd} = 94 \text{ kNm}$$

Buckling utilisation

$$M_{Ed} / M_{b,Rd} = 44 \%$$

OK

Deflection



Variable action deflection limit

$$\Delta_Q = 14.5 \text{ mm}$$

Variable action deflection

$$\delta_Q = 1.6 \text{ mm}$$

OK

Total deflection limit

$$\Delta_{G+Q} = 26.2 \text{ mm}$$

Total deflection

$$\delta_{G+Q} = 9.2 \text{ mm}$$

OK

Section properties

Elastic modulus - major axis, yy

$$W_{el} = 292 \text{ cm}^3$$

Plastic modulus - major axis, yy

$$W_{pl} = 342 \text{ cm}^3$$

Second moment of area - major axis, yy

$$I_y = 4,460 \text{ cm}^4$$

Second moment of area - minor axis, zz

$$I_z = 123 \text{ cm}^4$$

Warping constant

$$I_w = 0.027 \text{ dm}^6$$

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Torsional constant $I_t = 4.77 \text{ cm}^4$

Area of section $A = 3,160 \text{ mm}^2$

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

Young's modulus of elasticity $E = 210,000 \text{ N/mm}^2$ cl.3.2.6

Poisson's ratio in elastic stage $\nu = 0.3$ cl.3.2.6

Shear modulus $G_s = 81,000 \text{ N/mm}^2$ 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 7.00 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 $c_f / t_f = 5.76$

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 $c_w / t_w = 47.6$

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 $h_w = 291 \text{ mm}$

Shear area for I and H sections $A_v = 1,880 \text{ mm}^2$ cl.6.2.6 (3)

Design shear resistance $V_{pl,Rd} = 299 \text{ kN}$ 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 $h_w / t_w = 50.2$ 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 (32 kN) is less than half of the plastic shear resistance ($299 \text{ kN} / 2 = 150 \text{ kN}$), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus $W_{pl} = 342,000 \text{ mm}^3$

Design bending resistance $M_{c,Rd} = 94 \text{ kNm}$ eq (6.13)

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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	$L_{cr} = 750 \text{ mm}$	
Critical buckling moment	$M_{CR} = 684 \text{ kNm}$	NCCI SN003b-EN-EU
Class 1 section, therefore use plastic modulus	$W_{pl} = 342,000 \text{ mm}^3$	cl.6.3.2.1(3)
Non-dimensional slenderness ratio	$\bar{\lambda}_{LT} = 0.371$	cl.6.3.2.2 (1)
Depth to width ratio for buckling curve	$h / b = 3$	
Buckling curve for h / b ratio	Buckling curve = c	Table 6.5 / BS-EN NA
Imperfection factor for buckling curve c	$\alpha_{LT} = 0.49$	Table 6.3 / BS-EN NA
Intermediate factor for reduction factor calculation	$\phi_{LT} = 0.544$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = 1$	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	$\chi_{LT,mod} = 1$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = 94 \text{ kNm}$	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)

Beam No.5 Padstones Calculation

COMPLIES WITH LATEST EUROPEAN DESIGN CODES

Beam End Reaction = **31.60** kN (factored) Variable Load Safety Factor = 1.5
Factored Load at End of Beam Permanent Load Safety Factor = 1.35

Characteristic strength of masonry = **4.5** N/mm² (Brickwork usually = 4.5 N/mm²)
(3.6N Blockwork usually = 2.6 N/mm²)
Width of beam end bearing = **102** mm (A Engineering Brick = 13.2 N/mm²)
Length of beam end bearing = **150** mm (B Engineering Brick = 10.5 N/mm²)
(Weak Brickwork = approx 2.8 N/mm²)
(7.3N Blockwork usually = 4.2 N/mm²)
(10.4N Blockwork usually = 5.4 N/mm²)

$\gamma_m = 3.0$

Bearing Factor = **1.25**

Results

Maximum Bearing Stress = 1.88 N/mm²
Actual Bearing Stress = 2.07 N/mm²

Padstone Required

Padstone Results

Characteristic strength of Padstone = **10.5** N/mm² (A Engineering Brick = 13.2 N/mm²)
(B Engineering Brick = 10.5 N/mm²)
Width of Padstone = **100** mm (Concrete C15 = 15 N/mm²)
Length of Padstone = **215** mm (Concrete C30 = 30 N/mm²)
(Concrete C40 = 40 N/mm²)
(Steel Plate = 275 N/mm²)

Allowable padstone stress = 4.38 N/mm²
Stress under beam end bearing = 2.07 N/mm² Therefore Padstone Stress OK
Allowable masonry stress = 1.88 N/mm²
Stress under padstone = 1.47 N/mm² Therefore Masonry Stress OK

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Steel Beam Design

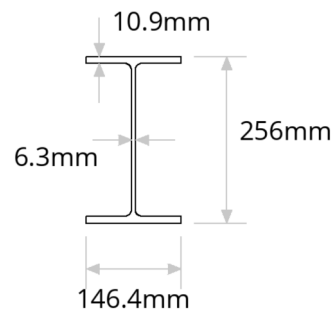
To Eurocode BS EN 1993-1-1/NA:2008

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	280 kN	27.7 kN	10 %	OK
Bending moment	133 kNm	33.1 kNm	25 %	OK
Buckling	66.5 kNm	33.1 kNm	50 %	OK
Total deflection	30.6 mm	8.3 mm	27 %	OK
Deflection due to variable actions	17 mm	3.3 mm	19 %	OK

Section details

Type	Universal beam
Section	254 x 146 x 37 UB
Steel grade	S275
Width	b = 146 mm
Depth	h = 256 mm
Web thickness	t _w = 6.3 mm
Flange thickness	t _f = 10.9 mm
Root radius	r = 7.6 mm
Mass per metre	w = 37 kg/m



Span and restraints

Effective span	L = 6,120 mm
Buckling length	L _{cr} = 6,120 mm

Deflection limits

Variable action deflection limit	$\Delta_Q = L / 360 = \mathbf{17 \text{ mm}}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = \mathbf{30.6 \text{ mm}}$

Safety factors

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

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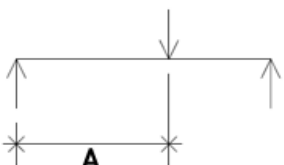
Loading details



Self weight

Permanent action

$$SW = w \times 9.81 / 1000 = \mathbf{0.363 \text{ kN/m}}$$



Load 1: Point load - PL from Beam No.4

Distance to point load, A

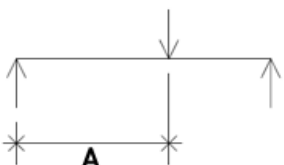
$$A = \mathbf{1.06 \text{ m}}$$

Permanent action

$$G_1 = \mathbf{4.86 \text{ kN}}$$

Variable action

$$Q_1 = \mathbf{4.01 \text{ kN}}$$



Load 2: Point load - PL from Beam No.4

Distance to point load, A

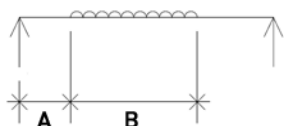
$$A = \mathbf{5.06 \text{ m}}$$

Permanent action

$$G_2 = \mathbf{4.86 \text{ kN}}$$

Variable action

$$Q_2 = \mathbf{4.01 \text{ kN}}$$



Load 3: Partial UDL - Flat roof, with no permanent access

Dimension to start of load, A

$$A = \mathbf{1.06 \text{ m}}$$

Length of load, B

$$B = \mathbf{4 \text{ m}}$$

Permanent action

$$G_3 = \mathbf{1 \text{ kN/m}^2 \times 1.4 \text{ m} = 1.4 \text{ kN/m}}$$

Variable action

$$Q_3 = \mathbf{0.75 \text{ kN/m}^2 \times 1.4 \text{ m} = 1.05 \text{ kN/m}}$$



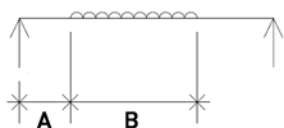
Load 4: UDL - 100mm Lightweight blockwork + Plaster or render on ONE side

Permanent action

$$G_4 = \mathbf{1 \text{ kN/m}^2 \times 0.225 \text{ m} = 0.225 \text{ kN/m}}$$

Variable action

$$Q_4 = \mathbf{0 \text{ kN/m}^2 \times 0.225 \text{ m} = 0 \text{ kN/m}}$$



Load 5: Partial UDL - Flat roof, with no permanent access

Dimension to start of load, A

$$A = \mathbf{5.06 \text{ m}}$$

Length of load, B

$$B = \mathbf{1.06 \text{ m}}$$

Permanent action

$$G_5 = \mathbf{1 \text{ kN/m}^2 \times 2.4 \text{ m} = 2.4 \text{ kN/m}}$$

Variable action

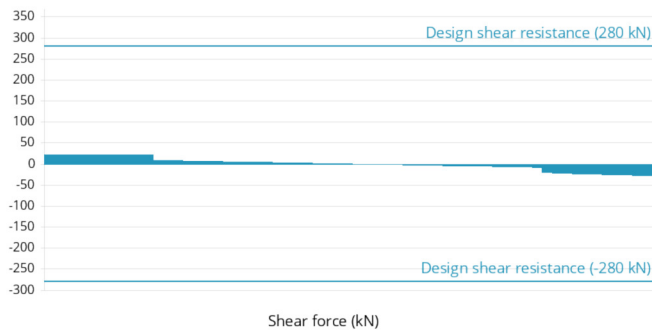
$$Q_5 = \mathbf{0.75 \text{ kN/m}^2 \times 2.4 \text{ m} = 1.8 \text{ kN/m}}$$

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	9.69 kN	6.28 kN	16 kN	22.5 kN
Right reaction	11.8 kN	7.85 kN	19.6 kN	27.7 kN

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Design shear force



Design shear force

$$V_{Ed} = 27.7 \text{ kN}$$

Design shear resistance

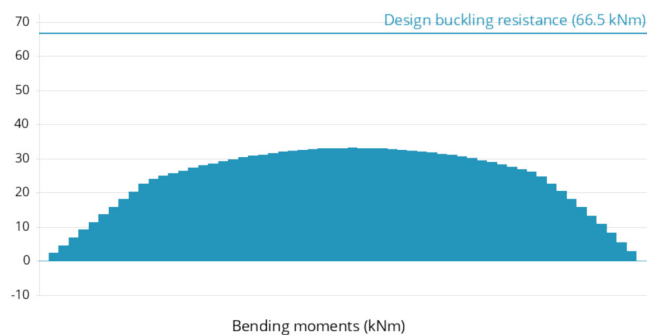
$$V_{c,Rd} = 280 \text{ kN}$$

Utilisation

$$V_{Ed} / V_{c,Rd} = 10 \%$$

OK

Design bending moment



Design bending moment, major axis

$$M_{Ed} = 33.1 \text{ kNm}$$

Design resistance for bending

$$M_{c,Rd} = 133 \text{ kNm}$$

Bending utilisation

$$M_{Ed} / M_{c,Rd} = 25 \%$$

OK

Design resistance for buckling

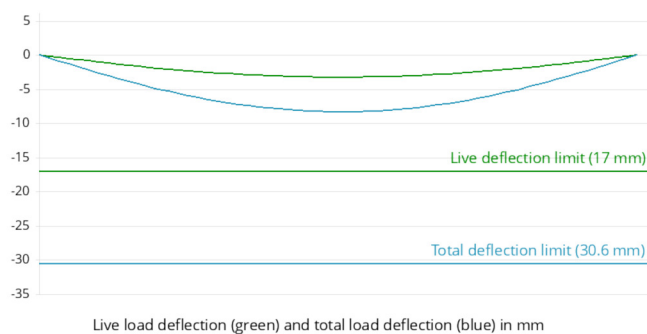
$$M_{b,Rd} = 66.5 \text{ kNm}$$

Buckling utilisation

$$M_{Ed} / M_{b,Rd} = 50 \%$$

OK

Deflection



Variable action deflection limit

$$\Delta_Q = 17 \text{ mm}$$

Variable action deflection

$$\delta_Q = 3.3 \text{ mm}$$

OK

Total deflection limit

$$\Delta_{G+Q} = 30.6 \text{ mm}$$

Total deflection

$$\delta_{G+Q} = 8.3 \text{ mm}$$

OK

Section properties

Elastic modulus - major axis, yy

$$W_{el} = 433 \text{ cm}^3$$

Plastic modulus - major axis, yy

$$W_{pl} = 483 \text{ cm}^3$$

Second moment of area - major axis, yy

$$I_y = 5,540 \text{ cm}^4$$

Second moment of area - minor axis, zz

$$I_z = 571 \text{ cm}^4$$

Warping constant

$$I_w = 0.0857 \text{ dm}^6$$

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Torsional constant $I_T = 15.3 \text{ cm}^4$

Area of section $A = 4,720 \text{ mm}^2$

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

Young's modulus of elasticity $E = 210,000 \text{ N/mm}^2$ cl.3.2.6

Poisson's ratio in elastic stage $\nu = 0.3$ cl.3.2.6

Shear modulus $G_s = 81,000 \text{ N/mm}^2$ 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 10.90 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 $c_f / t_f = 5.73$

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 $c_w / t_w = 34.8$

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 $h_w = 234 \text{ mm}$

Shear area for I and H sections $A_v = 1,760 \text{ mm}^2$ cl.6.2.6 (3)

Design shear resistance $V_{pl,Rd} = 280 \text{ kN}$ 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 $h_w / t_w = 37.2$ 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 (28 kN) is less than half of the plastic shear resistance ($280 \text{ kN} / 2 = 140 \text{ kN}$), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus $W_{pl} = 483,000 \text{ mm}^3$

Design bending resistance $M_{c,Rd} = 133 \text{ kNm}$ eq (6.13)

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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	$L_{cr} = 6,120 \text{ mm}$	
Critical buckling moment	$M_{CR} = 73.5 \text{ kNm}$	NCCI SN003b-EN-EU
Class 1 section, therefore use plastic modulus	$W_{pl} = 483,000 \text{ mm}^3$	cl.6.3.2.1(3)
Non-dimensional slenderness ratio	$\bar{\lambda}_{LT} = 1.34$	cl.6.3.2.2 (1)
Depth to width ratio for buckling curve	$h / b = 1.75$	
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	$\phi_{LT} = 1.34$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = 0.501$	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	$\chi_{LT,mod} = 0.501$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = 66.5 \text{ kNm}$	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)

Beam No.6 Padstones Calculation

COMPLIES WITH LATEST EUROPEAN DESIGN CODES

Beam End Reaction = **27.70** kN (factored) Variable Load Safety Factor = 1.5
Factored Load at End of Beam Permanent Load Safety Factor = 1.35

Characteristic strength of masonry = **4.2** N/mm² (Brickwork usually = 4.5 N/mm²)
(3.6N Blockwork usually = 2.6 N/mm²)
Width of beam end bearing = **102** mm (A Engineering Brick = 13.2 N/mm²)
Length of beam end bearing = **150** mm (B Engineering Brick = 10.5 N/mm²)
(Weak Brickwork = approx 2.8 N/mm²)
(7.3N Blockwork usually = 4.2 N/mm²)
(10.4N Blockwork usually = 5.4 N/mm²)

$\gamma_m = 3.0$

Bearing Factor = **1.25**

Results

Maximum Bearing Stress = 1.75 N/mm²
Actual Bearing Stress = 1.81 N/mm²

Padstone Required

Padstone Results

Characteristic strength of Padstone = **10.5** N/mm² (A Engineering Brick = 13.2 N/mm²)
(B Engineering Brick = 10.5 N/mm²)
Width of Padstone = **100** mm (Concrete C15 = 15 N/mm²)
Length of Padstone = **215** mm (Concrete C30 = 30 N/mm²)
(Concrete C40 = 40 N/mm²)
(Steel Plate = 275 N/mm²)

Allowable padstone stress = 4.38 N/mm²
Stress under beam end bearing = 1.81 N/mm² Therefore Padstone Stress OK
Allowable masonry stress = 1.75 N/mm²
Stress under padstone = 1.29 N/mm² Therefore Masonry Stress OK

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Steel Beam Design

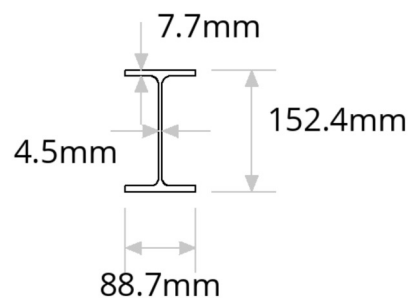
To Eurocode BS EN 1993-1-1/NA:2008

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	130 kN	11 kN	9 %	OK
Bending moment	33.8 kNm	7.88 kNm	23 %	OK
Buckling	22.5 kNm	7.88 kNm	35 %	OK
Total deflection	14.3 mm	2.8 mm	19 %	OK
Deflection due to variable actions	7.9 mm	0.5 mm	7 %	OK

Section details

Type	Universal beam
Section	152 x 89 x 16 UB
Steel grade	S275
Width	b = 88.7 mm
Depth	h = 152 mm
Web thickness	t _w = 4.5 mm
Flange thickness	t _f = 7.7 mm
Root radius	r = 7.6 mm
Mass per metre	w = 16 kg/m



Span and restraints

Effective span	L = 2,860 mm
Buckling length	L _{cr} = 2,860 mm

Deflection limits

Variable action deflection limit	$\Delta_Q = L / 360 = \mathbf{7.94 \text{ mm}}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = \mathbf{14.3 \text{ mm}}$

Safety factors

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

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Loading details



Self weight

Permanent action

$$SW = w \times 9.81 / 1000 = \mathbf{0.157 \text{ kN/m}}$$



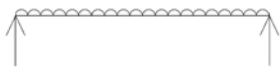
Load 1: UDL - 100mm Lightweight blockwork + Plaster or render on ONE side

Permanent action

$$G_1 = 1 \text{ kN/m}^2 \times 2.4 \text{ m} = \mathbf{2.4 \text{ kN/m}}$$

Variable action

$$Q_1 = 0 \text{ kN/m}^2 \times 2.4 \text{ m} = \mathbf{0 \text{ kN/m}}$$



Load 2: UDL - Ceiling beneath sloping roof

Permanent action

$$G_2 = 0.3 \text{ kN/m}^2 \times 0.8 \text{ m} = \mathbf{0.24 \text{ kN/m}}$$

Variable action

$$Q_2 = 0.25 \text{ kN/m}^2 \times 0.8 \text{ m} = \mathbf{0.2 \text{ kN/m}}$$



Load 3: UDL - Sloping roof, 30° to 45°

Permanent action

$$G_3 = 1.41 \text{ kN/m}^2 \times 1.2 \text{ m} = \mathbf{1.69 \text{ kN/m}}$$

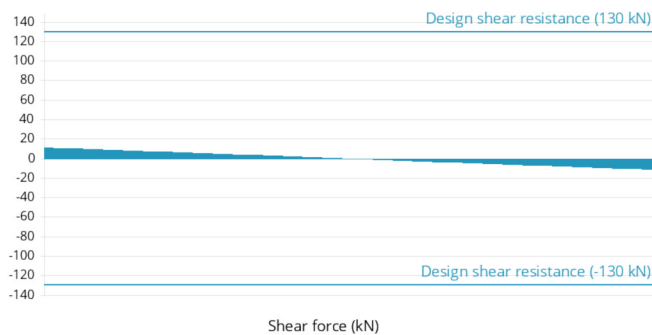
Variable action

$$Q_3 = 0.75 \text{ kN/m}^2 \times 1.2 \text{ m} = \mathbf{0.9 \text{ kN/m}}$$

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	6.42 kN	1.57 kN	7.99 kN	11 kN
Right reaction	6.42 kN	1.57 kN	7.99 kN	11 kN

Design shear force



Design shear force

$$V_{Ed} = \mathbf{11 \text{ kN}}$$

Design shear resistance

$$V_{c,Rd} = \mathbf{130 \text{ kN}}$$

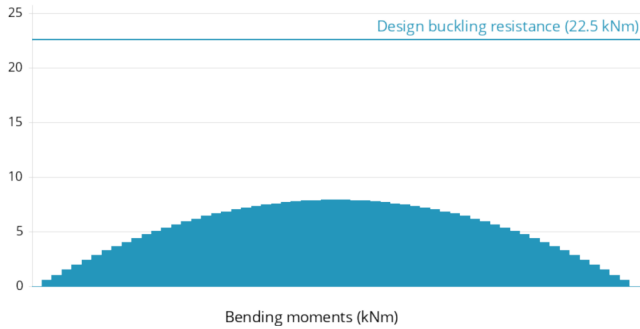
Utilisation

$$V_{Ed} / V_{c,Rd} = \mathbf{9 \%}$$

OK

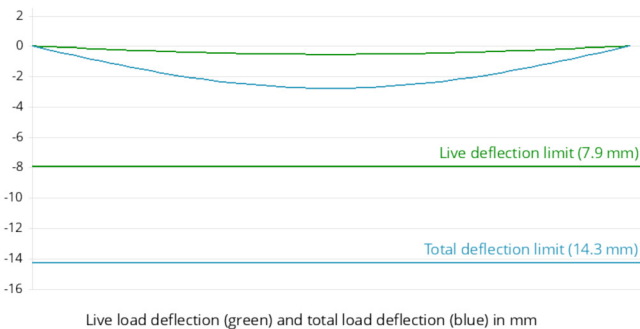
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Design bending moment



Design bending moment, major axis	$M_{Ed} = 7.88 \text{ kNm}$
Design resistance for bending	$M_{c,Rd} = 33.8 \text{ kNm}$
Bending utilisation	$M_{Ed} / M_{c,Rd} = 23 \% \text{ OK}$
Design resistance for buckling	$M_{b,Rd} = 22.5 \text{ kNm}$
Buckling utilisation	$M_{Ed} / M_{b,Rd} = 35 \% \text{ OK}$

Deflection



Variable action deflection limit	$\Delta_Q = 7.9 \text{ mm}$
Variable action deflection	$\delta_Q = 0.5 \text{ mm} \text{ OK}$
Total deflection limit	$\Delta_{G+Q} = 14.3 \text{ mm}$
Total deflection	$\delta_{G+Q} = 2.8 \text{ mm} \text{ OK}$

Section properties

Elastic modulus - major axis, yy	$W_{el} = 109 \text{ cm}^3$
Plastic modulus - major axis, yy	$W_{pl} = 123 \text{ cm}^3$
Second moment of area - major axis, yy	$I_y = 834 \text{ cm}^4$
Second moment of area - minor axis, zz	$I_z = 89.8 \text{ cm}^4$
Warping constant	$I_w = 0.0047 \text{ dm}^6$
Torsional constant	$I_t = 3.56 \text{ cm}^4$
Area of section	$A = 2,030 \text{ mm}^2$

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

Young's modulus of elasticity	$E = 210,000 \text{ N/mm}^2$	cl.3.2.6
Poisson's ratio in elastic stage	$\nu = 0.3$	cl.3.2.6
Shear modulus	$G_s = 81,000 \text{ N/mm}^2$	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 7.70 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

$$c_f / t_f = 4.48$$

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

$$c_w / t_w = 27.1$$

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

$$h_w = 137 \text{ mm}$$

Shear area for I and H sections

$$A_v = 816 \text{ mm}^2$$

cl.6.2.6 (3)

Design shear resistance

$$V_{pl,Rd} = 130 \text{ kN}$$

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

$$h_w / t_w = 30.4$$

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 (11 kN) is less than half of the plastic shear resistance ($130 \text{ kN} / 2 = 65 \text{ kN}$), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus

$$W_{pl} = 123,000 \text{ mm}^3$$

Design bending resistance

$$M_{c,Rd} = 33.8 \text{ kNm}$$

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

$$L_{cr} = 2,860 \text{ mm}$$

Critical buckling moment

$$M_{CR} = 30.4 \text{ kNm}$$

NCCI SN003b-EN-EU

Class 1 section, therefore use plastic modulus

$$W_{pl} = 123,000 \text{ mm}^3$$

cl.6.3.2.1(3)

Non-dimensional slenderness ratio

$$\bar{\lambda}_{LT} = 1.05$$

cl.6.3.2.2 (1)

Depth to width ratio for buckling curve

$$h / b = 1.72$$

Buckling curve for h / b ratio

$$\text{Buckling curve} = b$$

Table 6.5 / BS-EN NA

Imperfection factor for buckling curve b

$$\alpha_{LT} = 0.34$$

Table 6.3 / BS-EN NA

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Intermediate factor for reduction factor calculation	$\phi_{LT} = 1.03$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = 0.667$	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	$\chi_{LT,mod} = 0.667$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = 22.5 \text{ kNm}$	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)

Beam No.7 Padstones Calculation

COMPLIES WITH LATEST EUROPEAN DESIGN CODES

Beam End Reaction = **11.00** kN (factored) Variable Load Safety Factor = 1.5
Factored Load at End of Beam Permanent Load Safety Factor = 1.35

Characteristic strength of masonry = **4.2** N/mm² (Brickwork usually = 4.5 N/mm²)
(3.6N Blockwork usually = 2.6 N/mm²)
Width of beam end bearing = **89** mm (A Engineering Brick = 13.2 N/mm²)
Length of beam end bearing = **100** mm (B Engineering Brick = 10.5 N/mm²)
(Weak Brickwork = approx 2.8 N/mm²)
(7.3N Blockwork usually = 4.2 N/mm²)
(10.4N Blockwork usually = 5.4 N/mm²)

$\gamma_m = 3.0$

Bearing Factor = **1.25**

Results

Maximum Bearing Stress = 1.75 N/mm²
Actual Bearing Stress = 1.24 N/mm²

Padstone Not Required

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Steel Beam Design

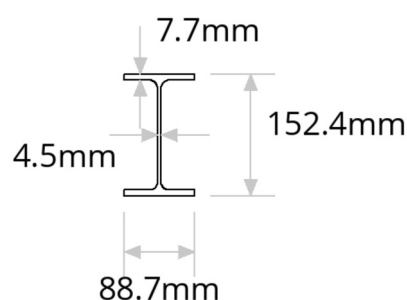
To Eurocode BS EN 1993-1-1/NA:2008

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	130 kN	6.83 kN	5 %	OK
Bending moment	33.8 kNm	4.88 kNm	14 %	OK
Buckling	22.5 kNm	4.88 kNm	22 %	OK
Total deflection	14.3 mm	1.7 mm	12 %	OK
Deflection due to variable actions	7.9 mm	0.2 mm	3 %	OK

Section details

Type	Universal beam
Section	152 x 89 x 16 UB
Steel grade	S275
Width	b = 88.7 mm
Depth	h = 152 mm
Web thickness	t _w = 4.5 mm
Flange thickness	t _f = 7.7 mm
Root radius	r = 7.6 mm
Mass per metre	w = 16 kg/m



Span and restraints

Effective span	L = 2,860 mm
Buckling length	L _{cr} = 2,860 mm

Deflection limits

Variable action deflection limit	$\Delta_Q = L / 360 = \mathbf{7.94 \text{ mm}}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = \mathbf{14.3 \text{ mm}}$

Safety factors

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

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Loading details



Self weight

Permanent action

$$SW = w \times 9.81 / 1000 = \mathbf{0.157 \text{ kN/m}}$$



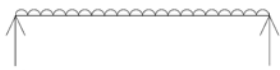
Load 1: UDL - 100mm Lightweight blockwork + Plaster or render on ONE side

Permanent action

$$G_1 = 1 \text{ kN/m}^2 \times 2.25 \text{ m} = \mathbf{2.25 \text{ kN/m}}$$

Variable action

$$Q_1 = 0 \text{ kN/m}^2 \times 2.25 \text{ m} = \mathbf{0 \text{ kN/m}}$$



Load 2: UDL - Ceiling beneath sloping roof

Permanent action

$$G_2 = 0.3 \text{ kN/m}^2 \times 0.4 \text{ m} = \mathbf{0.12 \text{ kN/m}}$$

Variable action

$$Q_2 = 0.25 \text{ kN/m}^2 \times 0.4 \text{ m} = \mathbf{0.1 \text{ kN/m}}$$



Load 3: UDL - Sloping roof, 30° to 45°

Permanent action

$$G_3 = 1.41 \text{ kN/m}^2 \times 0.4 \text{ m} = \mathbf{0.564 \text{ kN/m}}$$

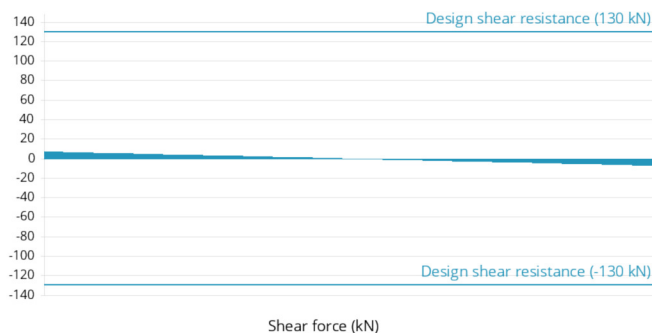
Variable action

$$Q_3 = 0.75 \text{ kN/m}^2 \times 0.4 \text{ m} = \mathbf{0.3 \text{ kN/m}}$$

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	4.42 kN	0.572 kN	4.99 kN	6.83 kN
Right reaction	4.42 kN	0.572 kN	4.99 kN	6.83 kN

Design shear force



Design shear force

$$V_{Ed} = \mathbf{6.83 \text{ kN}}$$

Design shear resistance

$$V_{c,Rd} = \mathbf{130 \text{ kN}}$$

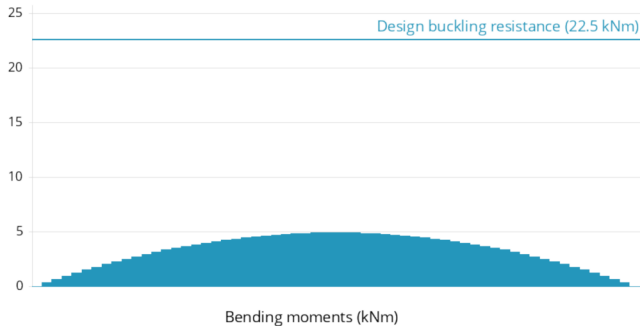
Utilisation

$$V_{Ed} / V_{c,Rd} = \mathbf{5 \%}$$

OK

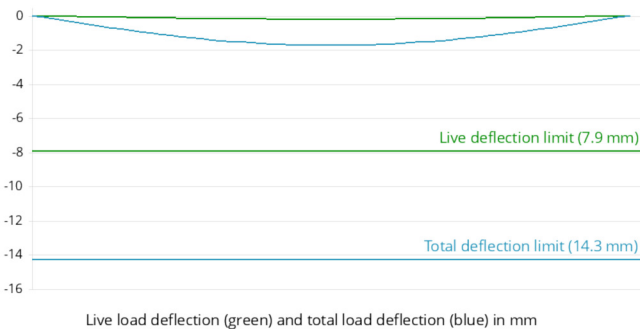
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Design bending moment



Design bending moment, major axis	$M_{Ed} = 4.88 \text{ kNm}$
Design resistance for bending	$M_{c,Rd} = 33.8 \text{ kNm}$
Bending utilisation	$M_{Ed} / M_{c,Rd} = 14 \% \text{ OK}$
Design resistance for buckling	$M_{b,Rd} = 22.5 \text{ kNm}$
Buckling utilisation	$M_{Ed} / M_{b,Rd} = 22 \% \text{ OK}$

Deflection



Variable action deflection limit	$\Delta_Q = 7.9 \text{ mm}$
Variable action deflection	$\delta_Q = 0.2 \text{ mm} \text{ OK}$
Total deflection limit	$\Delta_{G+Q} = 14.3 \text{ mm}$
Total deflection	$\delta_{G+Q} = 1.7 \text{ mm} \text{ OK}$

Section properties

Elastic modulus - major axis, yy	$W_{el} = 109 \text{ cm}^3$
Plastic modulus - major axis, yy	$W_{pl} = 123 \text{ cm}^3$
Second moment of area - major axis, yy	$I_y = 834 \text{ cm}^4$
Second moment of area - minor axis, zz	$I_z = 89.8 \text{ cm}^4$
Warping constant	$I_w = 0.0047 \text{ dm}^6$
Torsional constant	$I_t = 3.56 \text{ cm}^4$
Area of section	$A = 2,030 \text{ mm}^2$

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

Young's modulus of elasticity	$E = 210,000 \text{ N/mm}^2$	cl.3.2.6
Poisson's ratio in elastic stage	$\nu = 0.3$	cl.3.2.6
Shear modulus	$G_s = 81,000 \text{ N/mm}^2$	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 7.70 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 $c_f / t_f = 4.48$

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 $c_w / t_w = 27.1$

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 $h_w = 137 \text{ mm}$

Shear area for I and H sections $A_v = 816 \text{ mm}^2$ cl.6.2.6 (3)

Design shear resistance $V_{pl,Rd} = 130 \text{ kN}$ 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 $h_w / t_w = 30.4$ 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 (7 kN) is less than half of the plastic shear resistance ($130 \text{ kN} / 2 = 65 \text{ kN}$), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus $W_{pl} = 123,000 \text{ mm}^3$

Design bending resistance $M_{c,Rd} = 33.8 \text{ kNm}$ 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 $L_{cr} = 2,860 \text{ mm}$

Critical buckling moment $M_{CR} = 30.4 \text{ kNm}$ NCCI SN003b-EN-EU

Class 1 section, therefore use plastic modulus $W_{pl} = 123,000 \text{ mm}^3$ cl.6.3.2.1(3)

Non-dimensional slenderness ratio $\bar{\lambda}_{LT} = 1.05$ cl.6.3.2.2 (1)

Depth to width ratio for buckling curve $h / b = 1.72$

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

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Intermediate factor for reduction factor calculation	$\phi_{LT} = \mathbf{1.03}$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = \mathbf{0.667}$	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	$\chi_{LT,mod} = \mathbf{0.667}$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = \mathbf{22.5}$ 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)

Beam No.8 Padstones Calculation

COMPLIES WITH LATEST EUROPEAN DESIGN CODES

Beam End Reaction = **6.83** kN (factored) Variable Load Safety Factor = 1.5
Factored Load at End of Beam Permanent Load Safety Factor = 1.35

Characteristic strength of masonry = **2.6** N/mm² (Brickwork usually = 4.5 N/mm²)
(3.6N Blockwork usually = 2.6 N/mm²)
Width of beam end bearing = **89** mm (A Engineering Brick = 13.2 N/mm²)
Length of beam end bearing = **100** mm (B Engineering Brick = 10.5 N/mm²)
(Weak Brickwork = approx 2.8 N/mm²)
(7.3N Blockwork usually = 4.2 N/mm²)
(10.4N Blockwork usually = 5.4 N/mm²)

$\gamma_m = 3.0$

Bearing Factor = **1.25**

Results

Maximum Bearing Stress = 1.08 N/mm²
Actual Bearing Stress = 0.77 N/mm²

Padstone Not Required

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	Calcs for Beam No.9	Date 14 May 2022

Steel Beam Design

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

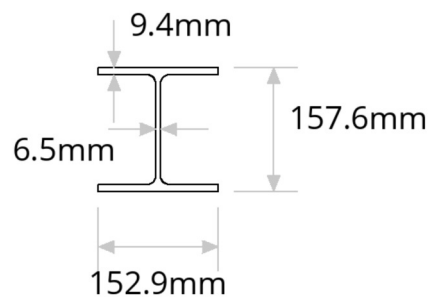
NOTE: Beam is to be sites with in the floor zone. Fix continuous timber rails to the web and suspend new and existing floor joists in new joist hangers fixed to the timber rails.

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	184 kN	26.8 kN	15 %	OK
Bending moment	68.2 kNm	27.9 kNm	41 %	OK
Buckling	53.1 kNm	27.9 kNm	52 %	OK
Total deflection	20.8 mm	9.6 mm	46 %	OK
Deflection due to variable actions	11.6 mm	5.3 mm	45 %	OK

Section details

Type	Universal column
Section	152 x 152 x 30 UC
Steel grade	S275
Width	b = 153 mm
Depth	h = 158 mm
Web thickness	t _w = 6.5 mm
Flange thickness	t _f = 9.4 mm
Root radius	r = 7.6 mm
Mass per metre	w = 30 kg/m



Span and restraints

Effective span	L = 4,160 mm
Buckling length	L _{cr} = 4,160 mm

Deflection limits

Variable action deflection limit	$\Delta_Q = L / 360 = \mathbf{11.6 \text{ mm}}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = \mathbf{20.8 \text{ mm}}$

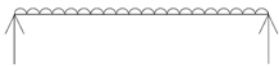
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Safety factors

Partial factor for permanent actions $\gamma_G = 1.35$

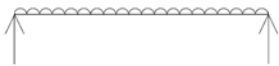
Partial factor for variable actions $\gamma_Q = 1.5$

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 $G_1 = 0.6 \text{ kN/m}^2 \times 2.82 \text{ m} = 1.7 \text{ kN/m}$

Variable action $Q_1 = 1.5 \text{ kN/m}^2 \times 2.82 \text{ m} = 4.24 \text{ kN/m}$



Load 2: UDL - Ceiling beneath sloping roof

Permanent action $G_2 = 0.3 \text{ kN/m}^2 \times 2.82 \text{ m} = 0.848 \text{ kN/m}$

Variable action $Q_2 = 0.25 \text{ kN/m}^2 \times 2.82 \text{ m} = 0.706 \text{ kN/m}$



Load 3: UDL - Timber Stud Wall + 2 Sheets of Plasterboard

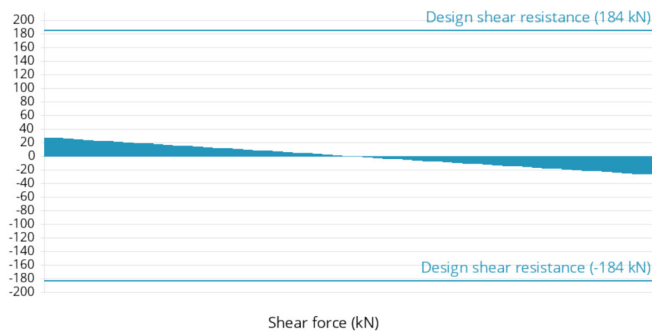
Permanent action $G_3 = 0.5 \text{ kN/m}^2 \times 2.4 \text{ m} = 1.2 \text{ kN/m}$

Variable action $Q_3 = 0 \text{ kN/m}^2 \times 2.4 \text{ m} = 0 \text{ kN/m}$

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	8.4 kN	10.3 kN	18.7 kN	26.8 kN
Right reaction	8.4 kN	10.3 kN	18.7 kN	26.8 kN

Design shear force



Design shear force

$$V_{Ed} = 26.8 \text{ kN}$$

Design shear resistance

$$V_{c,Rd} = 184 \text{ kN}$$

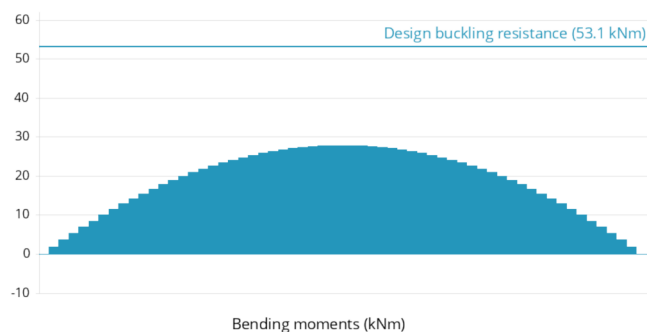
Utilisation

$$V_{Ed} / V_{c,Rd} = 15 \%$$

OK

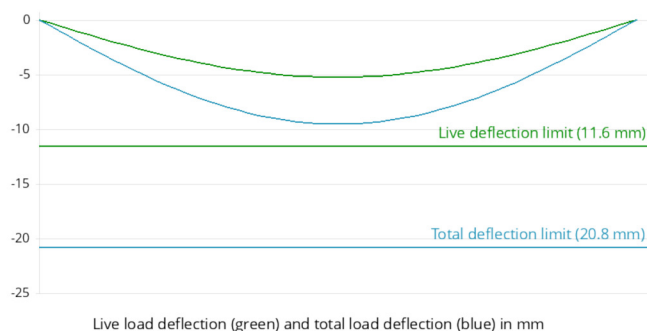
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Design bending moment



Design bending moment, major axis	$M_{Ed} = 27.9 \text{ kNm}$
Design resistance for bending	$M_{c,Rd} = 68.2 \text{ kNm}$
Bending utilisation	$M_{Ed} / M_{c,Rd} = 41 \% \text{ OK}$
Design resistance for buckling	$M_{b,Rd} = 53.1 \text{ kNm}$
Buckling utilisation	$M_{Ed} / M_{b,Rd} = 52 \% \text{ OK}$

Deflection



Variable action deflection limit	$\Delta_Q = 11.6 \text{ mm}$
Variable action deflection	$\delta_Q = 5.3 \text{ mm} \text{ OK}$
Total deflection limit	$\Delta_{G+Q} = 20.8 \text{ mm}$
Total deflection	$\delta_{G+Q} = 9.6 \text{ mm} \text{ OK}$

Section properties

Elastic modulus - major axis, yy	$W_{el} = 222 \text{ cm}^3$
Plastic modulus - major axis, yy	$W_{pl} = 248 \text{ cm}^3$
Second moment of area - major axis, yy	$I_y = 1,750 \text{ cm}^4$
Second moment of area - minor axis, zz	$I_z = 560 \text{ cm}^4$
Warping constant	$I_w = 0.031 \text{ dm}^6$
Torsional constant	$I_T = 10.5 \text{ cm}^4$
Area of section	$A = 3,830 \text{ mm}^2$

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

Young's modulus of elasticity	$E = 210,000 \text{ N/mm}^2$	cl.3.2.6
Poisson's ratio in elastic stage	$\nu = 0.3$	cl.3.2.6
Shear modulus	$G_s = 81,000 \text{ N/mm}^2$	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 9.40 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

$$c_f / t_f = 6.98$$

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

$$c_w / t_w = 19$$

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

$$h_w = 139 \text{ mm}$$

Shear area for I and H sections

$$A_v = 1,160 \text{ mm}^2$$

cl.6.2.6 (3)

Design shear resistance

$$V_{pl,Rd} = 184 \text{ kN}$$

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

$$h_w / t_w = 21.4$$

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 (27 kN) is less than half of the plastic shear resistance ($184 \text{ kN} / 2 = 92 \text{ kN}$), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus

$$W_{pl} = 248,000 \text{ mm}^3$$

Design bending resistance

$$M_{c,Rd} = 68.2 \text{ kNm}$$

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

$$L_{cr} = 4,160 \text{ mm}$$

Critical buckling moment

$$M_{CR} = 90.4 \text{ kNm}$$

NCCI SN003b-EN-EU

Class 1 section, therefore use plastic modulus

$$W_{pl} = 248,000 \text{ mm}^3$$

cl.6.3.2.1(3)

Non-dimensional slenderness ratio

$$\bar{\lambda}_{LT} = 0.868$$

cl.6.3.2.2 (1)

Depth to width ratio for buckling curve

$$h / b = 1.03$$

Buckling curve for h / b ratio

$$\text{Buckling curve} = b$$

Table 6.5 / BS-EN NA

Imperfection factor for buckling curve b

$$\alpha_{LT} = 0.34$$

Table 6.3 / BS-EN NA

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Intermediate factor for reduction factor calculation	$\phi_{LT} = \mathbf{0.862}$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = \mathbf{0.778}$	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	$\chi_{LT,mod} = \mathbf{0.778}$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = \mathbf{53.1}$ 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)

Beam No.9 Padstones Calculation

COMPLIES WITH LATEST EUROPEAN DESIGN CODES

Beam End Reaction = **26.80** kN (factored) Variable Load Safety Factor = 1.5
Factored Load at End of Beam Permanent Load Safety Factor = 1.35

Characteristic strength of masonry = **4.5** N/mm² (Brickwork usually = 4.5 N/mm²)
(3.6N Blockwork usually = 2.6 N/mm²)
Width of beam end bearing = **152** mm (A Engineering Brick = 13.2 N/mm²)
Length of beam end bearing = **100** mm (B Engineering Brick = 10.5 N/mm²)
(Weak Brickwork = approx 2.8 N/mm²)
(7.3N Blockwork usually = 4.2 N/mm²)
(10.4N Blockwork usually = 5.4 N/mm²)

$\gamma_m = 3.0$

Bearing Factor = **1.25**

Results

Maximum Bearing Stress = 1.88 N/mm²
Actual Bearing Stress = 1.76 N/mm²

Padstone Not Required

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Steel Beam Design

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

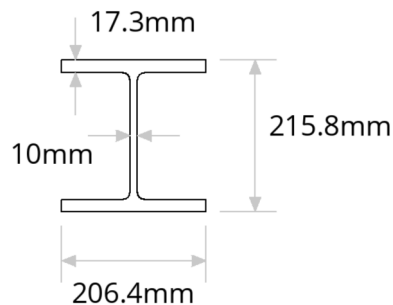
NOTE: Beam is to have a 10mm thick plate welded to the top flange to suit the thickness of the cavity wall beam supported.

Design summary

	Resistance / Limit	Applied / Actual	Utilisation	
Shear	371 kN	129 kN	35 %	OK
Bending moment	212 kNm	142 kNm	67 %	OK
Buckling	212 kNm	142 kNm	67 %	OK
Total deflection	20.5 mm	10.4 mm	51 %	OK
Deflection due to variable actions	11.4 mm	3.4 mm	30 %	OK

Section details

Type	Universal column
Section	203 x 203 x 71 UC
Steel grade	S275
Width	b = 206 mm
Depth	h = 216 mm
Web thickness	t _w = 10 mm
Flange thickness	t _f = 17.3 mm
Root radius	r = 10.2 mm
Mass per metre	w = 71 kg/m



Span and restraints

Effective span	L = 4,110 mm
Buckling length	L _{cr} = 400 mm

Deflection limits

Variable action deflection limit	$\Delta_Q = L / 360 = \mathbf{11.4 \text{ mm}}$
Total deflection limit	$\Delta_{G+Q} = L / 200 = \mathbf{20.5 \text{ mm}}$

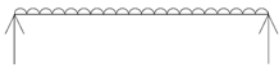
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Safety factors

Partial factor for permanent actions $\gamma_G = 1.35$

Partial factor for variable actions $\gamma_Q = 1.5$

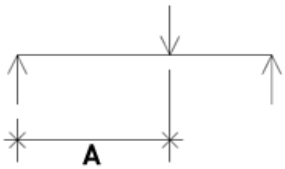
Loading details



Self weight

Permanent action

$$SW = w \times 9.81 / 1000 = \mathbf{0.697 \text{ kN/m}}$$



Load 1: Point load - PL from Loft Beams No.3

Distance to point load, A

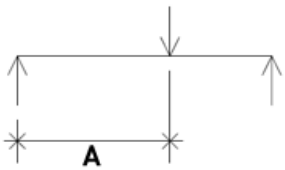
$$A = \mathbf{1.7 \text{ m}}$$

Permanent action

$$G_1 = \mathbf{27 \text{ kN}}$$

Variable action

$$Q_1 = \mathbf{24 \text{ kN}}$$



Load 2: Point load - PL from Loft Beam No.3

Distance to point load, A

$$A = \mathbf{3.55 \text{ m}}$$

Permanent action

$$G_2 = \mathbf{20 \text{ kN}}$$

Variable action

$$Q_2 = \mathbf{12 \text{ kN}}$$



Load 3: UDL - Cavity Wall

Permanent action

$$G_3 = \mathbf{4.9 \text{ kN/m}^2 \times 3.11 \text{ m} = 15.2 \text{ kN/m}}$$

Variable action

$$Q_3 = \mathbf{0 \text{ kN/m}^2 \times 3.11 \text{ m} = 0 \text{ kN/m}}$$



Load 4: UDL - Timber floor (domestic dwelling)

Permanent action

$$G_4 = \mathbf{0.6 \text{ kN/m}^2 \times 2.5 \text{ m} = 1.5 \text{ kN/m}}$$

Variable action

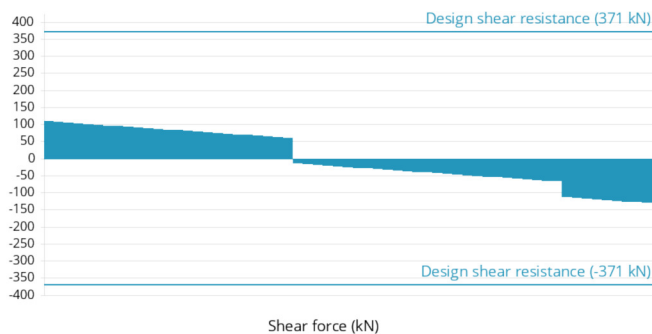
$$Q_4 = \mathbf{1.5 \text{ kN/m}^2 \times 2.5 \text{ m} = 3.75 \text{ kN/m}}$$

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	54.3 kN	23.4 kN	77.7 kN	108 kN
Right reaction	64.3 kN	28 kN	92.3 kN	129 kN

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Design shear force



Design shear force

$$V_{Ed} = 129 \text{ kN}$$

Design shear resistance

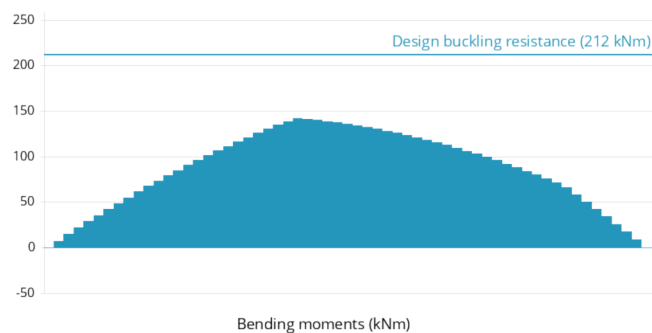
$$V_{c,Rd} = 371 \text{ kN}$$

Utilisation

$$V_{Ed} / V_{c,Rd} = 35 \%$$

OK

Design bending moment



Design bending moment, major axis

$$M_{Ed} = 142 \text{ kNm}$$

Design resistance for bending

$$M_{c,Rd} = 212 \text{ kNm}$$

Bending utilisation

$$M_{Ed} / M_{c,Rd} = 67 \%$$

OK

Design resistance for buckling

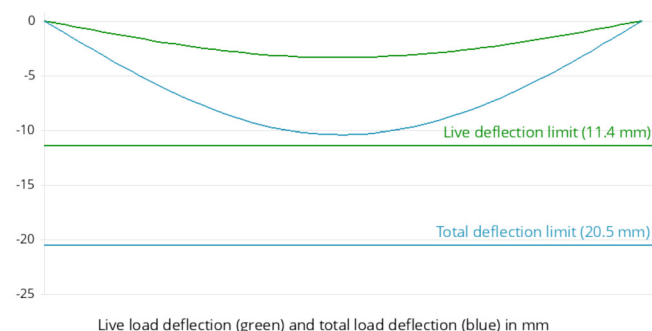
$$M_{b,Rd} = 212 \text{ kNm}$$

Buckling utilisation

$$M_{Ed} / M_{b,Rd} = 67 \%$$

OK

Deflection



Variable action deflection limit

$$\Delta_Q = 11.4 \text{ mm}$$

Variable action deflection

$$\delta_Q = 3.4 \text{ mm}$$

OK

Total deflection limit

$$\Delta_{G+Q} = 20.5 \text{ mm}$$

Total deflection

$$\delta_{G+Q} = 10.4 \text{ mm}$$

OK

Section properties

Elastic modulus - major axis, yy

$$W_{el} = 706 \text{ cm}^3$$

Plastic modulus - major axis, yy

$$W_{pl} = 799 \text{ cm}^3$$

Second moment of area - major axis, yy

$$I_y = 7,620 \text{ cm}^4$$

Second moment of area - minor axis, zz

$$I_z = 2,540 \text{ cm}^4$$

Warping constant

$$I_w = 0.25 \text{ dm}^6$$

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Torsional constant $I_T = 80.2 \text{ cm}^4$

Area of section $A = 9,040 \text{ mm}^2$

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

Young's modulus of elasticity $E = 210,000 \text{ N/mm}^2$ cl.3.2.6

Poisson's ratio in elastic stage $\nu = 0.3$ cl.3.2.6

Shear modulus $G_s = 81,000 \text{ N/mm}^2$ 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 17.30 mm $f_y = 265 \text{ N/mm}^2$ Tata blue book

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

Epsilon $\epsilon = 0.942$ EN 1993-1-1:2005 Table 5.2

Flange ratio for local buckling $c_f / t_f = 5.09$

Flange ratio limit for class 1 $9 \epsilon = 8.48$ Table 5.2 (sheet 2 of 3)

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

Web ratio for local buckling $c_w / t_w = 16.1$

Web ratio limit for class 1 $72 \epsilon = 67.8$ 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 $h_w = 181 \text{ mm}$

Shear area for I and H sections $A_v = 2,420 \text{ mm}^2$ cl.6.2.6 (3)

Design shear resistance $V_{pl,Rd} = 371 \text{ kN}$ 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 $h_w / t_w = 18.1$ EN 1993-1-5:2006 cl.5.1 (2)

Shear buckling limit $72 \epsilon / \eta = 67.8$ 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 (129 kN) is less than half of the plastic shear resistance ($371 \text{ kN} / 2 = 185 \text{ kN}$), therefore its effect on moment resistance may be neglected.

Class 1 section, therefore use plastic modulus $W_{pl} = 799,000 \text{ mm}^3$

Design bending resistance $M_{c,Rd} = 212 \text{ kNm}$ eq (6.13)

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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	$L_{cr} = 400 \text{ mm}$	
Critical buckling moment	$M_{CR} = 33,000 \text{ kNm}$	NCCI SN003b-EN-EU
Class 1 section, therefore use plastic modulus	$W_{pl} = 799,000 \text{ mm}^3$	cl.6.3.2.1(3)
Non-dimensional slenderness ratio	$\bar{\lambda}_{LT} = 0.0801$	cl.6.3.2.2 (1)
Depth to width ratio for buckling curve	$h / b = 1.05$	
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	$\phi_{LT} = 0.448$	cl.6.3.2.3 (1)
Buckling reduction factor	$\chi_{LT} = 1$	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	$\chi_{LT,mod} = 1$	eq (6.58)
Design buckling resistance	$M_{b,Rd} = 212 \text{ kNm}$	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)

Beam No.10 Padstones Calculation

COMPLIES WITH LATEST EUROPEAN DESIGN CODES

NOTE: Beam is to have minimum end-bearings of 200mm both ends and is to bear onto Class B engineering brick piers as shown on plan. Underpinning to be determined on site.

Beam End Reaction = **129.00** kN (factored) Variable Load Safety Factor = 1.5
Factored Load at End of Beam Permanent Load Safety Factor = 1.35

Characteristic strength of masonry = **10.5** N/mm² (Brickwork usually = 4.5 N/mm²)
(3.6N Blockwork usually = 2.6 N/mm²)
Width of beam end bearing = **206** mm (A Engineering Brick = 13.2 N/mm²)
Length of beam end bearing = **200** mm (B Engineering Brick = 10.5 N/mm²)
(Weak Brickwork = approx 2.8 N/mm²)
(7.3N Blockwork usually = 4.2 N/mm²)
(10.4N Blockwork usually = 5.4 N/mm²)

$\gamma_m = 3.0$

Bearing Factor = **1.25**

Results

Maximum Bearing Stress = 4.38 N/mm²
Actual Bearing Stress = 3.13 N/mm²

Padstone Not Required

Cavity Wall Lintel - CG90/100

Open back lintel

Benefits

Easy-to-use open back profile

Allows masonry to be built up continuously on both outer and inner leaf

Materials used in Lintels

The **CG**, **CH** and **CX** ranges are formed from galvanised steel, then powder coated

Duplex corrosion protection

Ensures optimum durability and longevity

Built-in DPC

Saves time in construction and means cavity is easy to clean without risk of damage to DPC

Integral Plaster key

With staggered slots applied to the inner flange and ribbed underside of insulation (perforated steel baseplate on **CH** and **CX** range)

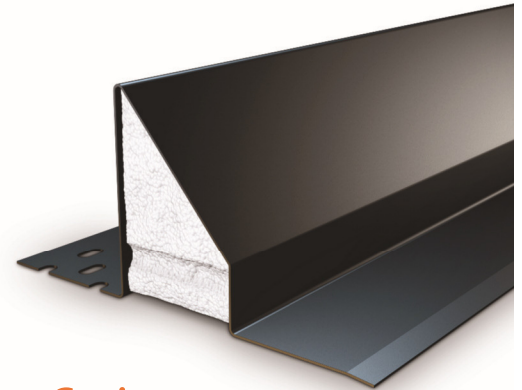
Continuous insulation

Maximising thermal efficiency, minimising cold bridging

Features:

- Triangulated masonry load
- Supporting uniformly distributed masonry load
- Supporting uniformly distributed timber floor and roof loads
- Suitable for fair faced inner leaf masonry

Note: Whilst the above information is intended to offer general guidance regarding typical applications, it should not be considered as comprehensive. Requirements not fully covered by the above should be referred to our technical services department for individual consideration.



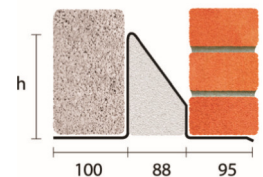
90-105mm Cavity 100-115mm Inner Leaf

All ratios are shown
inner to outer

Standard Duty



Standard lengths are available in 150mm increments up to 3000mm, 300mm at lengths from 3000mm to 3600mm.



CG90/100

Standard lengths (mm)	750-1500	1650-1800	1950-2100	2250-2400	2550-2700	2850-3600
SWL 1:1/3:1 (kN)	15	18	20	22	26	26
Weight (kg/m)	6.1	7.6	8.3	8.9	10.2	13.0
Nominal height 'h' (mm)	140	140	160	180	220	220

i The SWL (safe working load) is based on the total UDL (uniform distributed load) over maximum span using 150mm end bearings.

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Internal Wall Lintel - BSD100

Partition and loadbearing wall lintel

Benefits



Duplex corrosion protection

Ensures optimum durability and longevity



Integral Plaster key

With staggered holes to three sides of box profile

Universal application caters for all loading conditions:

- Direct floor or roof load
- Supports concrete floor loads
- Supports point loads e.g. steel beams

BSD, BHD and BXD

As previous plus:

- Suitable for 140mm blockwork



100mm

Interior Solid Walls

Standard Duty



Standard lengths are available in increments of 150mm at lengths up to 3000mm, 300mm at lengths from 3000mm to 4800mm (including 4575mm, but excluding 4500mm).



BSD100

Standard lengths (mm)	750-2100	2250-2700	2850-3600	3900-4575	4800
SWL (kN)	19	20	29	29	27
Weight (kg/m)	6.0	7.5	12.4	15.7	15.7
Nominal height 'h' (mm)	143	143	219	219	219



The SWL (safe working load) is based on the total UDL (uniform distributed load) over maximum span using 150mm end bearings.

Note: Whilst the above information is intended to offer general guidance regarding typical applications, it should not be considered as comprehensive. Requirements not fully covered by the above should be referred to our technical services department for individual consideration.

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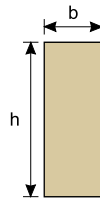
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	Calcs for Rafters to Front Porch Extension	Date 16 May 2022

Rafter Design to BS 5268-2 and BS 5268-7.5

Rafter details

Timber strength class	C16
Rafter width	b = 38 mm
Rafter depth	h = 95 mm
Rafter spacing	s = 400 mm
Rafter slope	$\alpha = 35$ degrees
Clear span on slope	$L_{cl,slope} = 0.9$ m
Brittle finishes (e.g. plasterboard) on the underside of the rafter?	No



Diagrams not to scale

Modification factors

Depth factor	$K_7 = (300 / h)^{0.11} = \mathbf{1.13}$
Load sharing modification factor (BS5268-2 clause 2.10.11)	$K_8 = \mathbf{1.10}$

Section properties

Mean modulus of elasticity	$E_{mean} = \mathbf{8,800}$ N/mm ²
Minimum modulus of elasticity	$E_{min} = \mathbf{5,800}$ N/mm ²
Cross section area	$A = b \times h = \mathbf{3,610}$ mm ²
Second moment of area	$I = b \times h^3 / 12 = \mathbf{2,720,000}$ mm ⁴
Section modulus	$Z = b \times h^2 / 6 = \mathbf{57,200}$ mm ³
Radius of gyration	$i = \sqrt{I / A} = \mathbf{27.4}$ mm
Average density for C16 timber (BS 5268-2:2002 Table 8)	$\rho_{mean} = \mathbf{370}$ kg/m ³
Self weight per linear metre ($g = 9.81$ m/s ²)	$F_{rafter} = b \times h \times \rho_{mean} \times g = \mathbf{0.0131}$ kN/m

Consider long term loading (0.75 kN/m² dead UDL. K3 = 1)

K3 (long term load)	$K_3 = \mathbf{1}$
Total load	$F = F_{dead} \times \cos(\alpha) \times (s / 1000) + F_{rafter} \times \cos(\alpha) = \mathbf{0.256}$ kN/m
Grade bending stress for C16 (BS5268-2:2002 Table 8)	$\sigma_{m,par} = \mathbf{5.3}$ N/mm ²
Permissible bending stress	$\sigma_{adm} = \sigma_{m,par} \times K_3 \times K_7 \times K_8 = \mathbf{6.62}$ N/mm ²
Compression perpendicular to grain for C16 (BS5268-2:2002 Table 8)	$\sigma_{c,per} = \mathbf{1.7}$ N/mm ²

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Notional bearing length

(Note from BS 5268-7.5 Clause 4.2: 'The bearing length required at each end of the rafter, calculated in accordance with 5.6, may not be sufficient for practical construction purposes.')

Effective span on slope

Check bending stress

Bending moment

Bending stress

$$a = (L_{cl,slope} \times F / 2) / (\sigma_{c,per} \times K_3 \times K_8 \times b - (F / 2)) = \mathbf{1.63 \text{ mm}}$$

$$L_{eff} = L_{cl,slope} + a = \mathbf{0.902 \text{ m}}$$

$$M = F \times L_{eff}^2 / 8 = \mathbf{0.0261 \text{ kNm}}$$

$$\sigma_{m,a} = (M \times 10^6) / Z = \mathbf{0.456 \text{ N/mm}^2}$$

$$\sigma_{m,a} \leq \sigma_{adm} \text{ (} \mathbf{0.456 \text{ N/mm}^2} \leq \mathbf{6.616 \text{ N/mm}^2} \text{) therefore OK}$$

Check shear stress

Grade shear stress for C16 (BS5268-2:2002 Table 8)

Permissible shear stress

Shear stress

$$\tau_{par} = \mathbf{0.67 \text{ N/mm}^2}$$

$$\tau_{adm} = \tau_{par} \times K_3 \times K_8 = \mathbf{0.737 \text{ N/mm}^2}$$

$$\tau = (3 \times F \times L_{eff} / 2 \times 10^3) / (2 \times b \times h) = \mathbf{0.048 \text{ N/mm}^2}$$

$$\tau \leq \tau_{adm} \text{ (} \mathbf{0.048 \text{ N/mm}^2} \leq \mathbf{0.737 \text{ N/mm}^2} \text{) therefore OK}$$

Check compressive stress parallel to grain

Compression stress parallel to grain

Minimum modulus of elasticity

Slenderness ratio

Compression member factor (calculated using equation in BS5268-7.5 clause 5.3.1)

Permissible compressive stress

Applied compressive stress

$$\sigma_{c,par} = \mathbf{6.8 \text{ N/mm}^2}$$

$$E_{min} = \mathbf{5,800 \text{ N/mm}^2}$$

$$\lambda = L_{eff} / i = \mathbf{32.9}$$

$$K_{12} = \mathbf{0.836}$$

$$\sigma_{c,adm} = \sigma_{c,par} \times K_3 \times K_8 \times K_{12} = \mathbf{6.25 \text{ N/mm}^2}$$

$$\sigma_{c,a} = \mathbf{0.113 \text{ N/mm}^2}$$

$$\sigma_{c,a} \leq \sigma_{c,adm} \text{ (} \mathbf{0.113 \text{ N/mm}^2} \leq \mathbf{6.254 \text{ N/mm}^2} \text{) therefore OK}$$

Check combined bending and compressive stress

Euler critical stress

Euler coefficient

Combined axial compression and bending check

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{53 \text{ N/mm}^2}$$

$$K_{eu} = 1 - (1.5 \times \sigma_c \times L_{12} / \sigma_e) = \mathbf{0.997}$$

$$= \sigma_{m,a} / (\sigma_{m,adm} \times K_{eu}) + \sigma_c / \sigma_{c,adm} = \mathbf{0.0872}$$

$$\mathbf{0.087 \leq 1 \text{ therefore OK}}$$

Check deflection

Permissible deflection

Bending deflection

Shear deflection

Total deflection

$$\delta_{adm} = 0.003 \times L_{eff} = \mathbf{2.7 \text{ mm}}$$

$$\delta_{bending} = (5 \times F \times L_{eff}^4) / (384 \times E_{mean} \times I) = \mathbf{0.0924 \text{ mm}}$$

$$\delta_{shear} = (12 \times F \times L_{eff}^2) / (5 \times E_{mean} \times b \times h) = \mathbf{0.0158 \text{ mm}}$$

$$\delta_{total} = \delta_{bending} + \delta_{shear} = \mathbf{0.108 \text{ mm}}$$

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$$\delta_{total} \leq \delta_{adm} \text{ (} 0.108 \text{ mm } \leq 2.705 \text{ mm) therefore OK}$$

Consider medium term loading (0.75 kN/m² dead UDL + 0.75 kN/m² imposed UDL. K3 = 1.25)

K3 (medium term load)

$$K_3 = 1.25$$

BS 5268-7.5 Clause 4.3: For a roof slope greater than 30° and not exceeding 75°: an imposed load obtained by linear interpolation between the values at 30° roof slope, e.g. 0.75 kN/m², and zero for a 75° roof slope.

Imposed load (UDL)

$$F_{imposed,udl} = 0.75 \times ((75 - \alpha) / 45) = 0.667 \text{ kN/m}^2$$

Total load

$$F = (F_{imposed,udl} \times \cos(\alpha)^2 + F_{dead} \times \cos(\alpha)) \times (s / 1000) + F_{rafter} \times \cos(\alpha) = 0.435 \text{ kN/m}$$

Grade bending stress for C16 (BS5268-2:2002 Table 8)

$$\sigma_{m,par} = 5.3 \text{ N/mm}^2$$

Permissible bending stress

$$\sigma_{adm} = \sigma_{m,par} \times K_3 \times K_7 \times K_8 = 8.27 \text{ N/mm}^2$$

Compression perpendicular to grain for C16 (BS5268-2:2002 Table 8)

$$\sigma_{c,per} = 1.7 \text{ N/mm}^2$$

Notional bearing length

$$a = (L_{cl,slope} \times F / 2) / (\sigma_{c,per} \times K_3 \times K_8 \times b - (F / 2)) = 2.21 \text{ mm}$$

(Note from BS 5268-7.5 Clause 4.2: 'The bearing length required at each end of the rafter, calculated in accordance with 5.6, may not be sufficient for practical construction purposes.')

Effective span on slope

$$L_{eff} = L_{cl,slope} + a = 0.902 \text{ m}$$

Check bending stress

Bending moment

$$M = F \times L_{eff}^2 / 8 = 0.0443 \text{ kNm}$$

Bending stress

$$\sigma_{m,a} = (M \times 10^6) / Z = 0.775 \text{ N/mm}^2$$

$$\sigma_{m,a} \leq \sigma_{adm} \text{ (} 0.775 \text{ N/mm}^2 \leq 8.27 \text{ N/mm}^2 \text{) therefore OK}$$

Check shear stress

Grade shear stress for C16 (BS5268-2:2002 Table 8)

$$\tau_{par} = 0.67 \text{ N/mm}^2$$

Permissible shear stress

$$\tau_{adm} = \tau_{par} \times K_3 \times K_8 = 0.921 \text{ N/mm}^2$$

Shear stress

$$\tau = (3 \times F \times L_{eff} / 2 \times 10^3) / (2 \times b \times h) = 0.0816 \text{ N/mm}^2$$

$$\tau \leq \tau_{adm} \text{ (} 0.082 \text{ N/mm}^2 \leq 0.921 \text{ N/mm}^2 \text{) therefore OK}$$

Check compressive stress parallel to grain

Compression stress parallel to grain

$$\sigma_{c,par} = 6.8 \text{ N/mm}^2$$

Minimum modulus of elasticity

$$E_{min} = 5,800 \text{ N/mm}^2$$

Slenderness ratio

$$\lambda = L_{eff} / i = 32.9$$

Compression member factor (calculated using equation in BS5268-7.5 clause 5.3.1)

$$K_{12} = 0.829$$

Permissible compressive stress

$$\sigma_{c,adm} = \sigma_{c,par} \times K_3 \times K_8 \times K_{12} = 7.76 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c,a} = 0.192 \text{ N/mm}^2$$

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$$\sigma_{c,a} \leq \sigma_{c,adm} \text{ (} 0.192 \text{ N/mm}^2 \leq 7.755 \text{ N/mm}^2 \text{) therefore OK}$$

Check combined bending and compressive stress

Euler critical stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 52.9 \text{ N/mm}^2$$

Euler coefficient

$$K_{eu} = 1 - (1.5 \times \sigma_c \times L_{12} / \sigma_e) = 0.995$$

Combined axial compression and bending check

$$= \sigma_{m,a} / (\sigma_{m,adm} \times K_{eu}) + \sigma_c / \sigma_{c,adm} = 0.119$$

$$0.119 \leq 1 \text{ therefore OK}$$

Check deflection

Permissible deflection

$$\delta_{adm} = 0.003 \times L_{eff} = 2.71 \text{ mm}$$

Bending deflection

$$\delta_{bending} = (5 \times F \times L_{eff}^4) / (384 \times E_{mean} \times I) = 0.157 \text{ mm}$$

Shear deflection

$$\delta_{shear} = (12 \times F \times L_{eff}^2) / (5 \times E_{mean} \times b \times h) = 0.0268 \text{ mm}$$

Total deflection

$$\delta_{total} = \delta_{bending} + \delta_{shear} = 0.184 \text{ mm}$$

$$\delta_{total} \leq \delta_{adm} \text{ (} 0.184 \text{ mm } \leq 2.707 \text{ mm) therefore OK}$$

Design summary

	Permissible	Applied/Actual	Utilisation	Result
Long term load shear stress (N/mm ²)	0.74	0.05	6.5 %	OK
Long term load bending stress (N/mm ²)	6.62	0.46	6.9 %	OK
Long term load deflection (mm)	2.7	0.11	4 %	OK
Long term compressive stress parallel to grain (N/mm ²)	6.25	0.11	1.8 %	OK
Long term combined bending and compressive stress (N/mm ²)	1	0.09	8.7 %	OK
Medium term load shear stress (N/mm ²)	0.92	0.08	8.9 %	OK
Medium term load bending stress (N/mm ²)	8.27	0.78	9.4 %	OK
Medium term load deflection (mm)	2.71	0.18	6.8 %	OK
Medium term compressive stress parallel to grain (N/mm ²)	7.76	0.19	2.5 %	OK
Medium term combined bending and compressive stress (N/mm ²)	1	0.12	11.9 %	OK

Notes

This design is in accordance with BS 5268-2:2002 Structural use of timber - Part 2: Code of practice for permissible stress design, materials and workmanship and BS 5268-7.5:1990 Structural use of timber - Section 7.5 Domestic rafters.

These calculations apply to systems of at least four rafters, and having tiling battens adequate to provide lateral distribution and lateral support.

Timber to be covered, this calculation is not to be used for timber which is fully exposed to the elements.

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As there are no brittle finishes e.g. plasterboard on the underside of the rafters the effects of deflection under the concentrated (point) load do not need to be considered as per guidance given in BS 5268-7.5 clause 4.3.

For roof slopes greater than 30 degrees, the concentrated (point) 0.9 kN load can be ignored in accordance with BS 5268-7.5 Clause 4.3.

These calculations are only applicable for roofs consisting of four or more rafters.

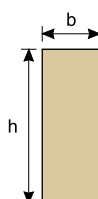
Wane as allowed in BS 4978:2007 + A2:2017 is permitted.

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	Calcs for Rafters to Front Roof Slope of the Proposed Two-Storey Extension	Date 16 May 2022

Rafter Design to BS 5268-2 and BS 5268-7.5

Rafter details

Timber strength class	C16
Rafter width	b = 47 mm
Rafter depth	h = 95 mm
Rafter spacing	s = 400 mm
Rafter slope	$\alpha = 35$ degrees
Clear span on slope	$L_{cl,slope} = 2.072$ m
Brittle finishes (e.g. plasterboard) on the underside of the rafter?	Yes



Diagrams not to scale

Modification factors

Depth factor	$K_7 = (300 / h)^{0.11} = 1.13$
Load sharing modification factor (BS5268-2 clause 2.10.11)	$K_8 = 1.10$

Section properties

Mean modulus of elasticity	$E_{mean} = 8,800$ N/mm²
Minimum modulus of elasticity	$E_{min} = 5,800$ N/mm²
Cross section area	$A = b \times h = 4,460$ mm²
Second moment of area	$I = b \times h^3 / 12 = 3,360,000$ mm⁴
Section modulus	$Z = b \times h^2 / 6 = 70,700$ mm³
Radius of gyration	$i = \sqrt{I / A} = 27.4$ mm
Average density for C16 timber (BS 5268-2:2002 Table 8)	$\rho_{mean} = 370$ kg/m³
Self weight per linear metre ($g = 9.81$ m/s ²)	$F_{rafter} = b \times h \times \rho_{mean} \times g = 0.0162$ kN/m

Consider long term loading (1 kN/m² dead UDL. $K_3 = 1$)

K_3 (long term load)	$K_3 = 1$
Total load	$F = F_{dead} \times \cos(\alpha) \times (s / 1000) + F_{rafter} \times \cos(\alpha) = 0.341$ kN/m
Grade bending stress for C16 (BS5268-2:2002 Table 8)	$\sigma_{m,par} = 5.3$ N/mm²
Permissible bending stress	$\sigma_{adm} = \sigma_{m,par} \times K_3 \times K_7 \times K_8 = 6.62$ N/mm²
Compression perpendicular to grain for C16 (BS5268-2:2002 Table 8)	$\sigma_{c,per} = 1.7$ N/mm²

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Notional bearing length

(Note from BS 5268-7.5 Clause 4.2: 'The bearing length required at each end of the rafter, calculated in accordance with 5.6, may not be sufficient for practical construction purposes.')

Effective span on slope

Check bending stress

Bending moment

Bending stress

$$a = (L_{cl,slope} \times F / 2) / (\sigma_{c,per} \times K_3 \times K_8 \times b - (F / 2)) = \mathbf{4.03 \text{ mm}}$$

$$L_{eff} = L_{cl,slope} + a = \mathbf{2.08 \text{ m}}$$

$$M = F \times L_{eff}^2 / 8 = \mathbf{0.184 \text{ kNm}}$$

$$\sigma_{m,a} = (M \times 10^6) / Z = \mathbf{2.6 \text{ N/mm}^2}$$

$$\sigma_{m,a} \leq \sigma_{adm} \text{ (} \mathbf{2.598 \text{ N/mm}^2} \leq \mathbf{6.616 \text{ N/mm}^2} \text{) therefore OK}$$

Check shear stress

Grade shear stress for C16 (BS5268-2:2002 Table 8)

Permissible shear stress

Shear stress

$$\tau_{par} = \mathbf{0.67 \text{ N/mm}^2}$$

$$\tau_{adm} = \tau_{par} \times K_3 \times K_8 = \mathbf{0.737 \text{ N/mm}^2}$$

$$\tau = (3 \times F \times L_{eff} / 2 \times 10^3) / (2 \times b \times h) = \mathbf{0.119 \text{ N/mm}^2}$$

$$\tau \leq \tau_{adm} \text{ (} \mathbf{0.119 \text{ N/mm}^2} \leq \mathbf{0.737 \text{ N/mm}^2} \text{) therefore OK}$$

Check compressive stress parallel to grain

Compression stress parallel to grain

Minimum modulus of elasticity

Slenderness ratio

Compression member factor (calculated using equation in BS5268-7.5 clause 5.3.1)

Permissible compressive stress

Applied compressive stress

$$\sigma_{c,par} = \mathbf{6.8 \text{ N/mm}^2}$$

$$E_{min} = \mathbf{5,800 \text{ N/mm}^2}$$

$$\lambda = L_{eff} / i = \mathbf{75.7}$$

$$K_{12} = \mathbf{0.542}$$

$$\sigma_{c,adm} = \sigma_{c,par} \times K_3 \times K_8 \times K_{12} = \mathbf{4.05 \text{ N/mm}^2}$$

$$\sigma_{c,a} = \mathbf{0.28 \text{ N/mm}^2}$$

$$\sigma_{c,a} \leq \sigma_{c,adm} \text{ (} \mathbf{0.28 \text{ N/mm}^2} \leq \mathbf{4.051 \text{ N/mm}^2} \text{) therefore OK}$$

Check combined bending and compressive stress

Euler critical stress

Euler coefficient

Combined axial compression and bending check

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{9.99 \text{ N/mm}^2}$$

$$K_{eu} = 1 - (1.5 \times \sigma_c \times L_{12} / \sigma_e) = \mathbf{0.977}$$

$$= \sigma_{m,a} / (\sigma_{m,adm} \times K_{eu}) + \sigma_c / \sigma_{c,adm} = \mathbf{0.471}$$

$$\mathbf{0.471 \leq 1 \text{ therefore OK}}$$

Check deflection

Permissible deflection

Bending deflection

Shear deflection

Total deflection

$$\delta_{adm} = 0.003 \times L_{eff} = \mathbf{6.23 \text{ mm}}$$

$$\delta_{bending} = (5 \times F \times L_{eff}^4) / (384 \times E_{mean} \times I) = \mathbf{2.79 \text{ mm}}$$

$$\delta_{shear} = (12 \times F \times L_{eff}^2) / (5 \times E_{mean} \times b \times h) = \mathbf{0.0898 \text{ mm}}$$

$$\delta_{total} = \delta_{bending} + \delta_{shear} = \mathbf{2.88 \text{ mm}}$$

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$$\delta_{total} \leq \delta_{adm} \text{ (} 2.88 \text{ mm } \leq 6.228 \text{ mm) therefore OK}$$

Consider medium term loading (1 kN/m² dead UDL + 0.75 kN/m² imposed UDL. K3 = 1.25)

K3 (medium term load)

$$K_3 = 1.25$$

BS 5268-7.5 Clause 4.3: For a roof slope greater than 30° and not exceeding 75°: an imposed load obtained by linear interpolation between the values at 30° roof slope, e.g. 0.75 kN/m², and zero for a 75° roof slope.

Imposed load (UDL)

$$F_{imposed,udl} = 0.75 \times ((75 - \alpha) / 45) = 0.667 \text{ kN/m}^2$$

Total load

$$F = (F_{imposed,udl} \times \cos(\alpha)^2 + F_{dead} \times \cos(\alpha)) \times (s / 1000) + F_{rafter} \times \cos(\alpha) = 0.52 \text{ kN/m}$$

Grade bending stress for C16 (BS5268-2:2002 Table 8)

$$\sigma_{m,par} = 5.3 \text{ N/mm}^2$$

Permissible bending stress

$$\sigma_{adm} = \sigma_{m,par} \times K_3 \times K_7 \times K_8 = 8.27 \text{ N/mm}^2$$

Compression perpendicular to grain for C16 (BS5268-2:2002 Table 8)

$$\sigma_{c,per} = 1.7 \text{ N/mm}^2$$

Notional bearing length

$$a = (L_{cl,slope} \times F / 2) / (\sigma_{c,per} \times K_3 \times K_8 \times b - (F / 2)) = 4.91 \text{ mm}$$

(Note from BS 5268-7.5 Clause 4.2: 'The bearing length required at each end of the rafter, calculated in accordance with 5.6, may not be sufficient for practical construction purposes.')

Effective span on slope

$$L_{eff} = L_{cl,slope} + a = 2.08 \text{ m}$$

Check bending stress

Bending moment

$$M = F \times L_{eff}^2 / 8 = 0.28 \text{ kNm}$$

Bending stress

$$\sigma_{m,a} = (M \times 10^6) / Z = 3.97 \text{ N/mm}^2$$

$$\sigma_{m,a} \leq \sigma_{adm} \text{ (} 3.965 \text{ N/mm}^2 \leq 8.27 \text{ N/mm}^2 \text{) therefore OK}$$

Check shear stress

Grade shear stress for C16 (BS5268-2:2002 Table 8)

$$\tau_{par} = 0.67 \text{ N/mm}^2$$

Permissible shear stress

$$\tau_{adm} = \tau_{par} \times K_3 \times K_8 = 0.921 \text{ N/mm}^2$$

Shear stress

$$\tau = (3 \times F \times L_{eff} / 2 \times 10^3) / (2 \times b \times h) = 0.181 \text{ N/mm}^2$$

$$\tau \leq \tau_{adm} \text{ (} 0.181 \text{ N/mm}^2 \leq 0.921 \text{ N/mm}^2 \text{) therefore OK}$$

Check compressive stress parallel to grain

Compression stress parallel to grain

$$\sigma_{c,par} = 6.8 \text{ N/mm}^2$$

Minimum modulus of elasticity

$$E_{min} = 5,800 \text{ N/mm}^2$$

Slenderness ratio

$$\lambda = L_{eff} / i = 75.7$$

Compression member factor (calculated using equation in BS5268-7.5 clause 5.3.1)

$$K_{12} = 0.494$$

Permissible compressive stress

$$\sigma_{c,adm} = \sigma_{c,par} \times K_3 \times K_8 \times K_{12} = 4.62 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c,a} = 0.427 \text{ N/mm}^2$$

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$$\sigma_{c,a} \leq \sigma_{c,adm} \text{ (} 0.427 \text{ N/mm}^2 \leq 4.616 \text{ N/mm}^2 \text{) therefore OK}$$

Check combined bending and compressive stress

Euler critical stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 9.98 \text{ N/mm}^2$$

Euler coefficient

$$K_{eu} = 1 - (1.5 \times \sigma_c \times L_{12} / \sigma_e) = 0.968$$

Combined axial compression and bending check

$$= \sigma_{m,a} / (\sigma_{m,adm} \times K_{eu}) + \sigma_c / \sigma_{c,adm} = 0.588$$

$$0.588 \leq 1 \text{ therefore OK}$$

Check deflection

Permissible deflection

$$\delta_{adm} = 0.003 \times L_{eff} = 6.23 \text{ mm}$$

Bending deflection

$$\delta_{bending} = (5 \times F \times L_{eff}^4) / (384 \times E_{mean} \times I) = 4.26 \text{ mm}$$

Shear deflection

$$\delta_{shear} = (12 \times F \times L_{eff}^2) / (5 \times E_{mean} \times b \times h) = 0.137 \text{ mm}$$

Total deflection

$$\delta_{total} = \delta_{bending} + \delta_{shear} = 4.4 \text{ mm}$$

$$\delta_{total} \leq \delta_{adm} \text{ (} 4.399 \text{ mm } \leq 6.231 \text{ mm) therefore OK}$$

Design summary

	Permissible	Applied/Actual	Utilisation	Result
Long term load shear stress (N/mm ²)	0.74	0.12	16.1 %	OK
Long term load bending stress (N/mm ²)	6.62	2.6	39.3 %	OK
Long term load deflection (mm)	6.23	2.88	46.2 %	OK
Long term compressive stress parallel to grain (N/mm ²)	4.05	0.28	6.9 %	OK
Long term combined bending and compressive stress (N/mm ²)	1	0.47	47.1 %	OK
Medium term load shear stress (N/mm ²)	0.92	0.18	19.7 %	OK
Medium term load bending stress (N/mm ²)	8.27	3.97	47.9 %	OK
Medium term load deflection (mm)	6.23	4.4	70.6 %	OK
Medium term compressive stress parallel to grain (N/mm ²)	4.62	0.43	9.2 %	OK
Medium term combined bending and compressive stress (N/mm ²)	1	0.59	58.8 %	OK

Notes

This design is in accordance with BS 5268-2:2002 Structural use of timber - Part 2: Code of practice for permissible stress design, materials and workmanship and BS 5268-7.5:1990 Structural use of timber - Section 7.5 Domestic rafters.

These calculations apply to systems of at least four rafters, and having tiling battens adequate to provide lateral distribution and lateral support.

Timber to be covered, this calculation is not to be used for timber which is fully exposed to the elements.

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For roof slopes greater than 30 degrees, the concentrated (point) 0.9 kN load can be ignored in accordance with BS 5268-7.5 Clause 4.3.

These calculations are only applicable for roofs consisting of four or more rafters.

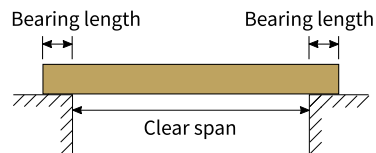
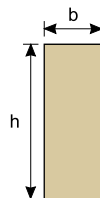
Wane as allowed in BS 4978:2007 + A2:2017 is permitted.

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Flat Roof Joist Design (BS5268-2:2002)

Joist details

Timber strength class	C16
Joist width	b = 47 mm
Joist depth	h = 195 mm
Joist spacing	s = 400 mm
Clear span	L_{cl} = 4.2 m



Diagrams not to scale

Modification factors

Depth factor	$K_7 = (300 / h)^{0.11} = \mathbf{1.05}$
Load sharing modification factor (BS5268-2 clause 2.10.11)	$K_8 = \mathbf{1.10}$

Section properties

Mean modulus of elasticity	$E_{\text{mean}} = \mathbf{8,800 \text{ N/mm}^2}$
Second moment of area	$I = b \times h^3 / 12 = \mathbf{29,000,000 \text{ mm}^4}$
Z to top edge of timber	$Z = b \times h^2 / 6 = \mathbf{298,000 \text{ mm}^3}$
Average density for C16 grade timber (BS 5268-2:2002 Table 8)	$\rho_{\text{mean}} = \mathbf{370 \text{ kg/m}^3}$
Joist self weight per square metre ($g = 9.81 \text{ m/s}^2$)	$F_{\text{joist}} = b \times h \times \rho_{\text{mean}} \times g / s = \mathbf{0.083 \text{ kN/m}^2}$

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Consider long term loading (0.75 kN/m² dead UDL. K3 = 1)

K3 (long term load)	$K_3 = 1$
Dead load (UDL)	$F_{\text{dead,udl}} = 0.75 \text{ kN/m}^2$
Total dead load (dead UDL + joist self weight)	$F_{\text{dead}} = F_{\text{dead,udl}} + F_{\text{joist}} = 0.833 \text{ kN/m}^2$
Total load per metre	$F = F_{\text{dead}} \times L_{\text{spacing}} = 0.333 \text{ kN/m}$
Compression perpendicular to grain for C16 (BS5268-2:2002 Table 8)	$\sigma_{\text{c,per}} = 1.7 \text{ N/mm}^2$
Notional bearing length (Note from BS 5268-7.1 Clause 4.2: 'The bearing length required at each end of the joist, calculated in accordance with 5.5, may not be sufficient for practical construction purposes.')	$a = (L_{\text{cl}} \times F / 2) / (\sigma_{\text{c,per}} \times K_3 \times K_8 \times b - (F / 2)) = 7.98 \text{ mm}$
Effective span	$L_{\text{eff}} = L_{\text{cl}} + a = 4.21 \text{ m}$
Check bending stress	
Grade bending stress for C16 (BS5268-2:2002 Table 8)	$\sigma_{\text{par}} = 5.3 \text{ N/mm}^2$
Permissible bending stress	$\sigma_{\text{adm}} = \sigma_{\text{par}} \times K_3 \times K_7 \times K_8 = 6.11 \text{ N/mm}^2$
Bending moment	$M = F \times L_{\text{eff}}^2 / 8 = 0.738 \text{ kNm}$
Bending stress	$\sigma = (M \times 10^6) / Z = 2.48 \text{ N/mm}^2$
	$\sigma \leq \sigma_{\text{adm}} (2.476 \text{ N/mm}^2 \leq 6.113 \text{ N/mm}^2) \text{ therefore OK}$

Check shear stress

Grade shear stress for C16 (BS5268-2:2002 Table 8)	$\tau_{\text{par}} = 0.67 \text{ N/mm}^2$
Permissible shear stress	$\tau_{\text{adm}} = \tau_{\text{par}} \times K_3 \times K_8 = 0.737 \text{ N/mm}^2$
Shear stress	$\tau = (3 \times F \times L_{\text{eff}} / 2 \times 10^3) / (2 \times b \times h) = 0.115 \text{ N/mm}^2$
	$\tau \leq \tau_{\text{adm}} (0.115 \text{ N/mm}^2 \leq 0.737 \text{ N/mm}^2) \text{ therefore OK}$

Check deflection

Permissible deflection	$\delta_{\text{adm}} = 0.003 \times L_{\text{eff}} = 12.6 \text{ mm}$
Bending deflection	$\delta_{\text{bending}} = (5 \times F \times L_{\text{eff}}^4) / (384 \times E_{\text{mean}} \times I) = 5.32 \text{ mm}$
Shear deflection	$\delta_{\text{shear}} = (12 \times F \times L_{\text{eff}}^2) / (5 \times E_{\text{mean}} \times b \times h) = 0.176 \text{ mm}$
Total deflection	$\delta_{\text{total}} = \delta_{\text{bending}} + \delta_{\text{shear}} = 5.5 \text{ mm}$
	$\delta_{\text{total}} \leq \delta_{\text{adm}} (5.499 \text{ mm} \leq 12.624 \text{ mm}) \text{ therefore OK}$

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Consider medium term loading (0.75 kN/m² dead UDL + 0.75 kN/m² live UDL. K₃ = 1.25)

K₃ (medium term load)

$$K_3 = \mathbf{1.25}$$

Dead load (UDL)

$$F_{\text{dead,udl}} = \mathbf{0.75 \text{ kN/m}^2}$$

Total dead load (dead UDL + joist self weight)

$$F_{\text{dead}} = F_{\text{dead,udl}} + F_{\text{joist}} = \mathbf{0.833 \text{ kN/m}^2}$$

Live load (UDL)

$$F_{\text{live,udl}} = \mathbf{0.75 \text{ kN/m}^2}$$

Total load per metre

$$F = (F_{\text{dead}} + F_{\text{live,udl}}) \times L_{\text{spacing}} = \mathbf{0.633 \text{ kN/m}}$$

Compression perpendicular to grain for C16
(BS5268-2:2002 Table 8)

$$\sigma_{\text{c,per}} = \mathbf{1.7 \text{ N/mm}^2}$$

Notional bearing length

$$a = (L_{\text{cl}} \times F / 2) / (\sigma_{\text{c,per}} \times K_3 \times K_8 \times b - (F / 2)) = \mathbf{12.1 \text{ mm}}$$

(Note from BS 5268-7.1 Clause 4.2: 'The bearing length required at each end of the joist, calculated in accordance with 5.5, may not be sufficient for practical construction purposes.')

Effective span

$$L_{\text{eff}} = L_{\text{cl}} + a = \mathbf{4.21 \text{ m}}$$

Check bending stress

Grade bending stress for C16 (BS5268-2:2002 Table 8)

$$\sigma_{\text{par}} = \mathbf{5.3 \text{ N/mm}^2}$$

Permissible bending stress

$$\sigma_{\text{adm}} = \sigma_{\text{par}} \times K_3 \times K_7 \times K_8 = \mathbf{7.64 \text{ N/mm}^2}$$

Bending moment

$$M = F \times L_{\text{eff}}^2 / 8 = \mathbf{1.4 \text{ kNm}}$$

Bending stress

$$\sigma = (M \times 10^6) / Z = \mathbf{4.72 \text{ N/mm}^2}$$

$$\sigma \leq \sigma_{\text{adm}} \text{ (} \mathbf{4.715 \text{ N/mm}^2 \leq 7.641 \text{ N/mm}^2 \text{) therefore OK}$$

Check shear stress

Grade shear stress for C16 (BS5268-2:2002 Table 8)

$$\tau_{\text{par}} = \mathbf{0.67 \text{ N/mm}^2}$$

Permissible shear stress

$$\tau_{\text{adm}} = \tau_{\text{par}} \times K_3 \times K_8 = \mathbf{0.921 \text{ N/mm}^2}$$

Shear stress

$$\tau = (3 \times F \times L_{\text{eff}} / 2 \times 10^3) / (2 \times b \times h) = \mathbf{0.218 \text{ N/mm}^2}$$

$$\tau \leq \tau_{\text{adm}} \text{ (} \mathbf{0.218 \text{ N/mm}^2 \leq 0.921 \text{ N/mm}^2 \text{) therefore OK}$$

Check deflection

Permissible deflection

$$\delta_{\text{adm}} = 0.003 \times L_{\text{eff}} = \mathbf{12.6 \text{ mm}}$$

Bending deflection

$$\delta_{\text{bending}} = (5 \times F \times L_{\text{eff}}^4) / (384 \times E_{\text{mean}} \times I) = \mathbf{10.2 \text{ mm}}$$

Shear deflection

$$\delta_{\text{shear}} = (12 \times F \times L_{\text{eff}}^2) / (5 \times E_{\text{mean}} \times b \times h) = \mathbf{0.334 \text{ mm}}$$

Total deflection

$$\delta_{\text{total}} = \delta_{\text{bending}} + \delta_{\text{shear}} = \mathbf{10.5 \text{ mm}}$$

$$\delta_{\text{total}} \leq \delta_{\text{adm}} \text{ (} \mathbf{10.491 \text{ mm} \leq 12.636 \text{ mm} \text{) therefore OK}$$

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Consider short term loading (0.75 kN/m² dead UDL + 0.9 kN live point load. K3 = 1.5)

K3 (short term load)

$$K_3 = 1.5$$

Dead load (UDL)

$$F_{\text{dead,udl}} = 0.75 \text{ kN/m}^2$$

Total dead load (dead UDL + joist self weight)

$$F_{\text{dead}} = F_{\text{dead,udl}} + F_{\text{joist}} = 0.833 \text{ kN/m}^2$$

Live point load

$$F_{\text{live,point}} = 0.9 \text{ kN}$$

Total load per metre

$$F = F_{\text{dead}} \times L_{\text{spacing}} = 0.333 \text{ kN/m}$$

Compression perpendicular to grain for C16
(BS5268-2:2002 Table 8)

$$\sigma_{\text{c,per}} = 1.7 \text{ N/mm}^2$$

Notional bearing length

(Note from BS 5268-7.1 Clause 4.2: 'The bearing length required at each end of the joist, calculated in accordance with 5.5, may not be sufficient for practical construction purposes.')

$$a = ((L_{\text{cl}} \times F / 2) + (F_{\text{live,point}} / 2)) / (\sigma_{\text{c,per}} \times K_3 \times K_8 \times b - (F / 2)) = 8.73 \text{ mm}$$

Effective span

$$L_{\text{eff}} = L_{\text{cl}} + a = 4.21 \text{ m}$$

Check bending stress

Grade bending stress for C16 (BS5268-2:2002 Table 8)

$$\sigma_{\text{par}} = 5.3 \text{ N/mm}^2$$

Permissible bending stress

$$\sigma_{\text{adm}} = \sigma_{\text{par}} \times K_3 \times K_7 \times K_8 = 9.17 \text{ N/mm}^2$$

Bending moment

$$M = (F \times L_{\text{eff}}^2 / 8) + (F_{\text{live,point}} \times L_{\text{eff}} / 4) = 1.68 \text{ kNm}$$

Bending stress

$$\sigma = (M \times 10^6) / Z = 5.66 \text{ N/mm}^2$$

$$\sigma \leq \sigma_{\text{adm}} (5.657 \text{ N/mm}^2 \leq 9.169 \text{ N/mm}^2) \text{ therefore OK}$$

Check shear stress

Grade shear stress for C16 (BS5268-2:2002 Table 8)

$$\tau_{\text{par}} = 0.67 \text{ N/mm}^2$$

Permissible shear stress

$$\tau_{\text{adm}} = \tau_{\text{par}} \times K_3 \times K_8 = 1.11 \text{ N/mm}^2$$

Shear stress

$$\tau = (3 \times ((F \times L_{\text{eff}} / 2) + F_{\text{live,point}}) \times 10^3) / (2 \times b \times h) = 0.262 \text{ N/mm}^2$$

$$\tau \leq \tau_{\text{adm}} (0.262 \text{ N/mm}^2 \leq 1.106 \text{ N/mm}^2) \text{ therefore OK}$$

Check deflection

Permissible deflection

$$\delta_{\text{adm}} = 0.003 \times L_{\text{eff}} = 12.6 \text{ mm}$$

Bending deflection from uniformly distributed load

$$\delta_{\text{bending,udl}} = (5 \times F \times L_{\text{eff}}^4) / (384 \times E_{\text{mean}} \times I) = 5.33 \text{ mm}$$

Shear deflection from uniformly distributed load

$$\delta_{\text{shear,udl}} = (12 \times F \times L_{\text{eff}}^2) / (5 \times E_{\text{mean}} \times b \times h) = 0.176 \text{ mm}$$

Bending deflection from point load

$$\delta_{\text{bending,point}} = (F_{\text{live,point}} \times 10^3 \times L_{\text{eff}}^3) / (48 \times E_{\text{mean}} \times I) = 5.47 \text{ mm}$$

Shear deflection from point load

$$\delta_{\text{shear,point}} = (24 \times F_{\text{live,point}} \times 10^3 \times L_{\text{eff}}) / (5 \times E_{\text{mean}} \times b \times h) = 0.225 \text{ mm}$$

Total deflection

$$\delta_{\text{total}} = \delta_{\text{bending,udl}} + \delta_{\text{shear,udl}} + \delta_{\text{bending,point}} + \delta_{\text{shear,point}} = 11.2 \text{ mm}$$

$$\delta_{\text{total}} \leq \delta_{\text{adm}} (11.198 \text{ mm} \leq 12.626 \text{ mm}) \text{ therefore OK}$$

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Design summary

	Permissible	Applied/Actual	Utilisation	Result
Long term load shear stress (N/mm ²)	0.74	0.11	15.6 %	OK
Long term load bending stress (N/mm ²)	6.11	2.48	40.5 %	OK
Long term load deflection (mm)	12.6	5.5	43.6 %	OK
Medium term load shear stress (N/mm ²)	0.92	0.22	23.7 %	OK
Medium term load bending stress (N/mm ²)	7.64	4.72	61.7 %	OK
Medium term load deflection (mm)	12.6	10.5	83 %	OK
Short term load shear stress (N/mm ²)	1.11	0.26	23.7 %	OK
Short term load bending stress (N/mm ²)	9.17	5.66	61.7 %	OK
Short term load deflection (mm)	12.6	11.2	88.7 %	OK

Notes

This design is in accordance with BS 5268-2:2002 Structural use of timber - Part 2: Code of practice for permissible stress design, materials and workmanship.

Timber to be covered, this calculation is not to be used for timber which is fully exposed to the elements.

The depth to width ratio of the timber does not exceed 6 and as per the requirements of BS 5268-2 Table 19 there is no risk of buckling under design load provided; The ends are held in position and compression edge held in line, as by direct connection of sheathing, deck or joists.

Notches are not to exceed 0.125 of the depth of the joist and are to be located between 0.07 and 0.25 of the span from the support.

Holes drilled at the neutral axis (centre line) are not to exceed 0.25 of the depth of a joist and are not to be less than three diameters (centre to centre) apart and are to be located between 0.25 and 0.4 of the span from the support.

Strutting or blocking between joists is required as follows;

- Joist spans up to 2.5m - no strutting required
- Joist spans 2.5m to 4.5m - provide strutting at mid-span
- Joist spans over 4.5m - provide 2 rows of strutting at third span positions

Solid timber strutting to be at least 38mm thick and be a minimum of three quarters of the joist depth.

Proprietary strutting to be used in accordance with manufacturers recommendations.

The joists are to have a minimum end bearing of 40mm.

Wane as allowed in BS 4978:2007 + A2:2017 is permitted.

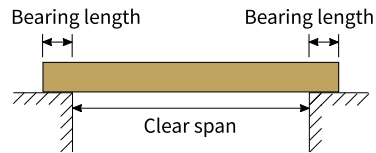
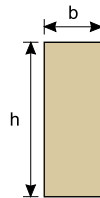
These calculations are only applicable for flat roofs consisting of four or more joists.

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	Calcs for Proposed First-Floor Level Floor Joists to Two-Storey Side Extension	Date 3 Nov 2021

Domestic Floor Joist Design (BS5268-2:2002)

Joist details

Timber strength class	C16
Joist width	b = 47 mm
Joist depth	h = 170 mm
Joist spacing	s = 400 mm
Clear span	L_{cl} = 2.76 m



Diagrams not to scale

Modification factors

Depth factor	$K_7 = (300 / h)^{0.11} = \mathbf{1.06}$
Load sharing modification factor (BS5268-2 clause 2.10.11)	$K_8 = \mathbf{1.10}$

Section properties

Mean modulus of elasticity	$E_{\text{mean}} = \mathbf{8,800 \text{ N/mm}^2}$
Second moment of area	$I = b \times h^3 / 12 = \mathbf{19,200,000 \text{ mm}^4}$
Z to top edge of timber	$Z = b \times h^2 / 6 = \mathbf{226,000 \text{ mm}^3}$
Average density for C16 grade timber (BS 5268-2:2002 Table 8)	$\rho_{\text{mean}} = \mathbf{370 \text{ kg/m}^3}$
Joist self weight per square metre ($g = 9.81 \text{ m/s}^2$)	$F_{\text{joist}} = b \times h \times \rho_{\text{mean}} \times g / s = \mathbf{0.073 \text{ kN/m}^2}$

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Consider long term loading (0.75 kN/m² dead UDL + 1.5 kN/m² live UDL. K3 = 1)

K3 (long term load)	$K_3 = 1$
Dead load (UDL)	$F_{\text{dead,udl}} = 0.75 \text{ kN/m}^2$
Total dead load (dead UDL + joist self weight)	$F_{\text{dead}} = F_{\text{dead,udl}} + F_{\text{joist}} = 0.823 \text{ kN/m}^2$
Live load (UDL)	$F_{\text{live,udl}} = 1.5 \text{ kN/m}^2$
Total load per metre	$F = (F_{\text{dead}} + F_{\text{live,udl}}) \times L_{\text{spacing}} = 0.929 \text{ kN/m}$
Compression perpendicular to grain for C16 (BS5268-2:2002 Table 8)	$\sigma_{\text{c,per}} = 1.7 \text{ N/mm}^2$
Notional bearing length (Note from BS 5268-7.1 Clause 4.2: 'The bearing length required at each end of the joist, calculated in accordance with 5.5, may not be sufficient for practical construction purposes.'))	$a = (L_{\text{cl}} \times F / 2) / (\sigma_{\text{c,per}} \times K_3 \times K_8 \times b - (F / 2)) = 14.7 \text{ mm}$
Effective span	$L_{\text{eff}} = L_{\text{cl}} + a = 2.77 \text{ m}$
Check bending stress	
Grade bending stress for C16 (BS5268-2:2002 Table 8)	$\sigma_{\text{par}} = 5.3 \text{ N/mm}^2$
Permissible bending stress	$\sigma_{\text{adm}} = \sigma_{\text{par}} \times K_3 \times K_7 \times K_8 = 6.21 \text{ N/mm}^2$
Bending moment	$M = F \times L_{\text{eff}}^2 / 8 = 0.894 \text{ kNm}$
Bending stress	$\sigma = (M \times 10^6) / Z = 3.95 \text{ N/mm}^2$
	$\sigma \leq \sigma_{\text{adm}} (3.949 \text{ N/mm}^2 \leq 6.206 \text{ N/mm}^2) \text{ therefore OK}$

Check shear stress

Grade shear stress for C16 (BS5268-2:2002 Table 8)	$\tau_{\text{par}} = 0.67 \text{ N/mm}^2$
Permissible shear stress	$\tau_{\text{adm}} = \tau_{\text{par}} \times K_3 \times K_8 = 0.737 \text{ N/mm}^2$
Shear stress	$\tau = (3 \times F \times L_{\text{eff}} / 2 \times 10^3) / (2 \times b \times h) = 0.242 \text{ N/mm}^2$
	$\tau \leq \tau_{\text{adm}} (0.242 \text{ N/mm}^2 \leq 0.737 \text{ N/mm}^2) \text{ therefore OK}$

Check deflection

Permissible deflection	$\delta_{\text{adm}} = 0.003 \times L_{\text{eff}} \text{ (or max of 14mm)} = 8.32 \text{ mm}$
Bending deflection	$\delta_{\text{bending}} = (5 \times F \times L_{\text{eff}}^4) / (384 \times E_{\text{mean}} \times I) = 4.23 \text{ mm}$
Shear deflection	$\delta_{\text{shear}} = (12 \times F \times L_{\text{eff}}^2) / (5 \times E_{\text{mean}} \times b \times h) = 0.244 \text{ mm}$
Total deflection	$\delta_{\text{total}} = \delta_{\text{bending}} + \delta_{\text{shear}} = 4.48 \text{ mm}$
	$\delta_{\text{total}} \leq \delta_{\text{adm}} (4.478 \text{ mm} \leq 8.324 \text{ mm}) \text{ therefore OK}$

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Consider medium term loading (0.75 kN/m² dead UDL + 1.4 kN live point load. K3 = 1.25)

K3 (medium term load)

$$K_3 = \mathbf{1.25}$$

Dead load (UDL)

$$F_{\text{dead,udl}} = \mathbf{0.75 \text{ kN/m}^2}$$

Total dead load (dead UDL + joist self weight)

$$F_{\text{dead}} = F_{\text{dead,udl}} + F_{\text{joist}} = \mathbf{0.823 \text{ kN/m}^2}$$

Live point load

$$F_{\text{live,point}} = \mathbf{1.4 \text{ kN}}$$

Total load per metre

$$F = F_{\text{dead}} \times L_{\text{spacing}} = \mathbf{0.329 \text{ kN/m}}$$

Compression perpendicular to grain for C16
(BS5268-2:2002 Table 8)

$$\sigma_{\text{c,per}} = \mathbf{1.7 \text{ N/mm}^2}$$

Notional bearing length

(Note from BS 5268-7.1 Clause 4.2: 'The bearing length required at each end of the joist, calculated in accordance with 5.5, may not be sufficient for practical construction purposes.')

$$a = ((L_{\text{cl}} \times F / 2) + (F_{\text{live,point}} / 2)) / (\sigma_{\text{c,per}} \times K_3 \times K_8 \times b - (F / 2)) = \mathbf{10.5 \text{ mm}}$$

Effective span

$$L_{\text{eff}} = L_{\text{cl}} + a = \mathbf{2.77 \text{ m}}$$

Check bending stress

Grade bending stress for C16 (BS5268-2:2002 Table 8)

$$\sigma_{\text{par}} = \mathbf{5.3 \text{ N/mm}^2}$$

Permissible bending stress

$$\sigma_{\text{adm}} = \sigma_{\text{par}} \times K_3 \times K_7 \times K_8 = \mathbf{7.76 \text{ N/mm}^2}$$

Bending moment

$$M = (F \times L_{\text{eff}}^2 / 8) + (F_{\text{live,point}} \times L_{\text{eff}} / 4) = \mathbf{1.29 \text{ kNm}}$$

Bending stress

$$\sigma = (M \times 10^6) / Z = \mathbf{5.68 \text{ N/mm}^2}$$

$$\sigma \leq \sigma_{\text{adm}} \text{ (} \mathbf{5.678 \text{ N/mm}^2 \leq 7.757 \text{ N/mm}^2 \text{) therefore OK}$$

Check shear stress

Grade shear stress for C16 (BS5268-2:2002 Table 8)

$$\tau_{\text{par}} = \mathbf{0.67 \text{ N/mm}^2}$$

Permissible shear stress

$$\tau_{\text{adm}} = \tau_{\text{par}} \times K_3 \times K_8 = \mathbf{0.921 \text{ N/mm}^2}$$

Shear stress

$$\tau = (3 \times ((F \times L_{\text{eff}} / 2) + F_{\text{live,point}}) \times 10^3) / (2 \times b \times h) = \mathbf{0.348 \text{ N/mm}^2}$$

$$\tau \leq \tau_{\text{adm}} \text{ (} \mathbf{0.348 \text{ N/mm}^2 \leq 0.921 \text{ N/mm}^2 \text{) therefore OK}$$

Check deflection

Permissible deflection

$$\delta_{\text{adm}} = 0.003 \times L_{\text{eff}} \text{ (or max of 14mm)} = \mathbf{8.31 \text{ mm}}$$

Bending deflection from uniformly distributed load

$$\delta_{\text{bending,udl}} = (5 \times F \times L_{\text{eff}}^4) / (384 \times E_{\text{mean}} \times I) = \mathbf{1.49 \text{ mm}}$$

Shear deflection from uniformly distributed load

$$\delta_{\text{shear,udl}} = (12 \times F \times L_{\text{eff}}^2) / (5 \times E_{\text{mean}} \times b \times h) = \mathbf{0.0862 \text{ mm}}$$

Bending deflection from point load

$$\delta_{\text{bending,point}} = (F_{\text{live,point}} \times 10^3 \times L_{\text{eff}}^3) / (48 \times E_{\text{mean}} \times I) = \mathbf{3.66 \text{ mm}}$$

Shear deflection from point load

$$\delta_{\text{shear,point}} = (24 \times F_{\text{live,point}} \times 10^3 \times L_{\text{eff}}) / (5 \times E_{\text{mean}} \times b \times h) = \mathbf{0.265 \text{ mm}}$$

Total deflection

$$\delta_{\text{total}} = \delta_{\text{bending,udl}} + \delta_{\text{shear,udl}} + \delta_{\text{bending,point}} + \delta_{\text{shear,point}} = \mathbf{5.5 \text{ mm}}$$

$$\delta_{\text{total}} \leq \delta_{\text{adm}} \text{ (} \mathbf{5.504 \text{ mm} \leq 8.312 \text{ mm} \text{) therefore OK}$$

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Design summary

	Permissible	Applied/Actual	Utilisation	Result
Long term load shear stress (N/mm ²)	0.74	0.24	32.8 %	OK
Long term load bending stress (N/mm ²)	6.21	3.95	63.6 %	OK
Long term load deflection (mm)	8.32	4.48	53.8 %	OK
Medium term load shear stress (N/mm ²)	0.92	0.35	37.8 %	OK
Medium term load bending stress (N/mm ²)	7.76	5.68	73.2 %	OK
Medium term load deflection (mm)	8.31	5.5	66.2 %	OK

Notes

This design is in accordance with BS 5268-2:2002 Structural use of timber - Part 2: Code of practice for permissible stress design, materials and workmanship.

Timber to be covered, this calculation is not to be used for timber which is fully exposed to the elements.

The depth to width ratio of the timber does not exceed 6 and as per the requirements of BS 5268-2 Table 19 there is no risk of buckling under design load provided; The ends are held in position and compression edge held in line, as by direct connection of sheathing, deck or joists.

Notches are not to exceed 0.125 of the depth of the joist and are to be located between 0.07 and 0.25 of the span from the support.

Holes drilled at the neutral axis (centre line) are not to exceed 0.25 of the depth of a joist and are not to be less than three diameters (centre to centre) apart and are to be located between 0.25 and 0.4 of the span from the support.

Strutting or blocking between joists is required as follows;

- Joist spans up to 2.5m - no strutting required
- Joist spans 2.5m to 4.5m - provide strutting at mid-span
- Joist spans over 4.5m - provide 2 rows of strutting at third span positions

Solid timber strutting to be at least 38mm thick and be a minimum of three quarters of the joist depth.

Proprietary strutting to be used in accordance with manufacturers recommendations.

The joists are to have a minimum end bearing of 40mm.

Wane as allowed in BS 4978:2007 + A2:2017 is permitted.

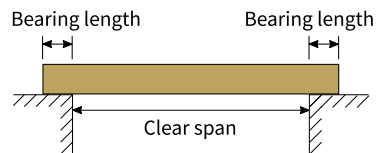
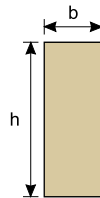
These calculations are only applicable for floors consisting of four or more joists.

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Ceiling Joist Design (BS5268-2:2002)

Joist details

Timber strength class	C16
Joist width	b = 38 mm
Joist depth	h = 95 mm
Joist spacing	s = 400 mm
Clear span	L_{cl} = 0.74 m



Diagrams not to scale

Modification factors

Depth factor	$K_7 = (300 / h)^{0.11} = \mathbf{1.13}$
Load sharing modification factor (BS5268-2 clause 2.10.11)	$K_8 = \mathbf{1.10}$

Section properties

Mean modulus of elasticity	$E_{\text{mean}} = \mathbf{8,800 \text{ N/mm}^2}$
Second moment of area	$I = b \times h^3 / 12 = \mathbf{2,720,000 \text{ mm}^4}$
Z to top edge of timber	$Z = b \times h^2 / 6 = \mathbf{57,200 \text{ mm}^3}$
Average density for C16 grade timber (BS 5268-2:2002 Table 8)	$\rho_{\text{mean}} = \mathbf{370 \text{ kg/m}^3}$
Joist self weight per square metre ($g = 9.81 \text{ m/s}^2$)	$F_{\text{joist}} = b \times h \times \rho_{\text{mean}} \times g / s = \mathbf{0.033 \text{ kN/m}^2}$

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Consider long term loading (0.25 kN/m² dead UDL + 0.25 kN/m² live UDL. K3 = 1)

K3 (long term load)	$K_3 = 1$
Dead load (UDL)	$F_{dead,udl} = 0.25 \text{ kN/m}^2$
Total dead load (dead UDL + joist self weight)	$F_{dead} = F_{dead,udl} + F_{joist} = 0.283 \text{ kN/m}^2$
Live load (UDL)	$F_{live,udl} = 0.25 \text{ kN/m}^2$
Total load per metre	$F = (F_{dead} + F_{live,udl}) \times L_{spacing} = 0.213 \text{ kN/m}$
Compression perpendicular to grain for C16 (BS5268-2:2002 Table 8)	$\sigma_{c,per} = 1.7 \text{ N/mm}^2$
Notional bearing length (Note from BS 5268-7.1 Clause 4.2: 'The bearing length required at each end of the joist, calculated in accordance with 5.5, may not be sufficient for practical construction purposes.')	$a = (L_{cl} \times F / 2) / (\sigma_{c,per} \times K_3 \times K_8 \times b - (F / 2)) = 1.11 \text{ mm}$
Effective span	$L_{eff} = L_{cl} + a = 0.741 \text{ m}$
Check bending stress	
Grade bending stress for C16 (BS5268-2:2002 Table 8)	$\sigma_{par} = 5.3 \text{ N/mm}^2$
Permissible bending stress	$\sigma_{adm} = \sigma_{par} \times K_3 \times K_7 \times K_8 = 6.62 \text{ N/mm}^2$
Bending moment	$M = F \times L_{eff}^2 / 8 = 0.0146 \text{ kNm}$
Bending stress	$\sigma = (M \times 10^6) / Z = 0.256 \text{ N/mm}^2$
	$\sigma \leq \sigma_{adm} \text{ (} 0.256 \text{ N/mm}^2 \leq 6.616 \text{ N/mm}^2 \text{) therefore OK}$

Check shear stress

Grade shear stress for C16 (BS5268-2:2002 Table 8)	$\tau_{par} = 0.67 \text{ N/mm}^2$
Permissible shear stress	$\tau_{adm} = \tau_{par} \times K_3 \times K_8 = 0.737 \text{ N/mm}^2$
Shear stress	$\tau = (3 \times F \times L_{eff} / 2 \times 10^3) / (2 \times b \times h) = 0.0328 \text{ N/mm}^2$
	$\tau \leq \tau_{adm} \text{ (} 0.033 \text{ N/mm}^2 \leq 0.737 \text{ N/mm}^2 \text{) therefore OK}$

Check deflection

Permissible deflection	$\delta_{adm} = 0.003 \times L_{eff} = 2.22 \text{ mm}$
Bending deflection	$\delta_{bending} = (5 \times F \times L_{eff}^4) / (384 \times E_{mean} \times I) = 0.035 \text{ mm}$
Shear deflection	$\delta_{shear} = (12 \times F \times L_{eff}^2) / (5 \times E_{mean} \times b \times h) = 0.00884 \text{ mm}$
Total deflection	$\delta_{total} = \delta_{bending} + \delta_{shear} = 0.0439 \text{ mm}$
	$\delta_{total} \leq \delta_{adm} \text{ (} 0.044 \text{ mm } \leq 2.223 \text{ mm) therefore OK}$

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Consider short term loading (0.25 kN/m² dead UDL + 0.25 kN/m² live UDL + 0.9 kN live point load. K3 = 1.5)

K3 (short term load)

$$K_3 = 1.5$$

Dead load (UDL)

$$F_{\text{dead,udl}} = 0.25 \text{ kN/m}^2$$

Total dead load (dead UDL + joist self weight)

$$F_{\text{dead}} = F_{\text{dead,udl}} + F_{\text{joist}} = 0.283 \text{ kN/m}^2$$

Live load (UDL)

$$F_{\text{live,udl}} = 0.25 \text{ kN/m}^2$$

Live point load

$$F_{\text{live,point}} = 0.9 \text{ kN}$$

Total load per metre

$$F = (F_{\text{dead}} + F_{\text{live,udl}}) \times L_{\text{spacing}} = 0.213 \text{ kN/m}$$

Compression perpendicular to grain for C16
(BS5268-2:2002 Table 8)

$$\sigma_{\text{c,per}} = 1.7 \text{ N/mm}^2$$

Notional bearing length

(Note from BS 5268-7.1 Clause 4.2: 'The bearing length required at each end of the joist, calculated in accordance with 5.5, may not be sufficient for practical construction purposes.')

$$a = ((L_{\text{cl}} \times F / 2) + (F_{\text{live,point}} / 2)) / (\sigma_{\text{c,per}} \times K_3 \times K_8 \times b - (F / 2)) = 4.97 \text{ mm}$$

Effective span

$$L_{\text{eff}} = L_{\text{cl}} + a = 0.745 \text{ m}$$

Check bending stress

Grade bending stress for C16 (BS5268-2:2002 Table 8)

$$\sigma_{\text{par}} = 5.3 \text{ N/mm}^2$$

Permissible bending stress

$$\sigma_{\text{adm}} = \sigma_{\text{par}} \times K_3 \times K_7 \times K_8 = 9.92 \text{ N/mm}^2$$

Bending moment

$$M = (F \times L_{\text{eff}}^2 / 8) + (F_{\text{live,point}} \times L_{\text{eff}} / 4) = 0.182 \text{ kNm}$$

Bending stress

$$\sigma = (M \times 10^6) / Z = 3.19 \text{ N/mm}^2$$

$$\sigma \leq \sigma_{\text{adm}} \text{ (} 3.191 \text{ N/mm}^2 \leq 9.924 \text{ N/mm}^2 \text{) therefore OK}$$

Check shear stress

Grade shear stress for C16 (BS5268-2:2002 Table 8)

$$\tau_{\text{par}} = 0.67 \text{ N/mm}^2$$

Permissible shear stress

$$\tau_{\text{adm}} = \tau_{\text{par}} \times K_3 \times K_8 = 1.11 \text{ N/mm}^2$$

Shear stress

$$\tau = (3 \times ((F \times L_{\text{eff}} / 2) + F_{\text{live,point}}) \times 10^3) / (2 \times b \times h) = 0.407 \text{ N/mm}^2$$

$$\tau \leq \tau_{\text{adm}} \text{ (} 0.407 \text{ N/mm}^2 \leq 1.106 \text{ N/mm}^2 \text{) therefore OK}$$

Check deflection

Permissible deflection

$$\delta_{\text{adm}} = 0.003 \times L_{\text{eff}} = 2.23 \text{ mm}$$

Bending deflection from uniformly distributed load

$$\delta_{\text{bending,udl}} = (5 \times F \times L_{\text{eff}}^4) / (384 \times E_{\text{mean}} \times I) = 0.0358 \text{ mm}$$

Shear deflection from uniformly distributed load

$$\delta_{\text{shear,udl}} = (12 \times F \times L_{\text{eff}}^2) / (5 \times E_{\text{mean}} \times b \times h) = 0.00893 \text{ mm}$$

Bending deflection from point load

$$\delta_{\text{bending,point}} = (F_{\text{live,point}} \times 10^3 \times L_{\text{eff}}^3) / (48 \times E_{\text{mean}} \times I) = 0.324 \text{ mm}$$

Shear deflection from point load

$$\delta_{\text{shear,point}} = (24 \times F_{\text{live,point}} \times 10^3 \times L_{\text{eff}}) / (5 \times E_{\text{mean}} \times b \times h) = 0.101 \text{ mm}$$

Total deflection

$$\delta_{\text{total}} = \delta_{\text{bending,udl}} + \delta_{\text{shear,udl}} + \delta_{\text{bending,point}} + \delta_{\text{shear,point}} = 0.47 \text{ mm}$$

$$\delta_{\text{total}} \leq \delta_{\text{adm}} \text{ (} 0.47 \text{ mm } \leq 2.235 \text{ mm) therefore OK}$$

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Design summary

	Permissible	Applied/Actual	Utilisation	Result
Long term load shear stress (N/mm ²)	0.74	0.03	4.5 %	OK
Long term load bending stress (N/mm ²)	6.62	0.26	3.9 %	OK
Long term load deflection (mm)	2.22	0.04	2 %	OK
Short term load shear stress (N/mm ²)	1.11	0.41	36.8 %	OK
Short term load bending stress (N/mm ²)	9.92	3.19	32.2 %	OK
Short term load deflection (mm)	2.23	0.47	21.1 %	OK

Notes

This design is in accordance with BS 5268-2:2002 Structural use of timber - Part 2: Code of practice for permissible stress design, materials and workmanship.

Timber to be covered, this calculation is not to be used for timber which is fully exposed to the elements.

If the depth to width ratio is greater than 4, boarding is to be fastened on top of the ceiling joists.

The depth to width ratio of the timber does not exceed 6 and as per the requirements of BS 5268-2 Table 19 there is no risk of buckling under design load provided; The ends are held in position and compression edge held in line, as by direct connection of sheathing, deck or joists.

Notches are not to exceed 0.125 of the depth of the joist and are to be located between 0.07 and 0.25 of the span from the support.

Holes drilled at the neutral axis (centre line) are not to exceed 0.25 of the depth of a joist and are not to be less than three diameters (centre to centre) apart and are to be located between 0.25 and 0.4 of the span from the support.

Strutting or blocking between joists is required as follows;

- Joist spans up to 2.5m - no strutting required
- Joist spans 2.5m to 4.5m - provide strutting at mid-span
- Joist spans over 4.5m - provide 2 rows of strutting at third span positions

Solid timber strutting to be at least 38mm thick and be a minimum of three quarters of the joist depth.

Proprietary strutting to be used in accordance with manufacturers recommendations.

The joists are to have a minimum end bearing of 40mm.

Wane as allowed in BS 4978:2007 + A2:2017 is permitted.

These calculations are only applicable for ceilings consisting of four or more joists.