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| | Project | Project ref |
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Steel Beam Design

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

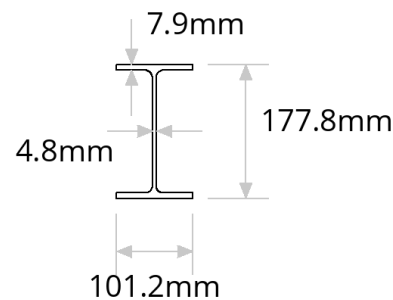
* You can add your own text, diagrams and photos here *

Design summary

| | Resistance / Limit | Applied / Actual | Utilisation | |
|------------------------------------|--------------------|------------------|-------------|-----------|
| Shear | 157 kN | 36.4 kN | 23 % | OK |
| Bending moment | 47 kNm | 27.3 kNm | 58 % | OK |
| Buckling | 31.6 kNm | 27.3 kNm | 86 % | OK |
| Total deflection | 15 mm | 6.4 mm | 42 % | OK |
| Deflection due to variable actions | 8.3 mm | 2.6 mm | 31 % | 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 = 3,000 mm |
| Buckling length | L _{cr} = 3,000 mm |

Deflection limits

| | |
|----------------------------------|---------------------------------------------------|
| Variable action deflection limit | $\Delta_Q = L / 360 = \mathbf{8.33 \text{ mm}}$ |
| Total deflection limit | $\Delta_{G+Q} = L / 200 = \mathbf{15 \text{ mm}}$ |

Safety factors

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

Loading details

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Self weight

Permanent action

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



Load 1: UDL - Ceiling beneath sloping roof

Permanent action

$$G_1 = \mathbf{0.3 \text{ kN/m}^2} \times \mathbf{3.5 \text{ m}} = \mathbf{1.05 \text{ kN/m}}$$

Variable action

$$Q_1 = \mathbf{0.25 \text{ kN/m}^2} \times \mathbf{3.5 \text{ m}} = \mathbf{0.875 \text{ kN/m}}$$



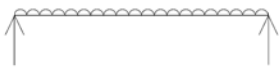
Load 2: UDL - 102.5mm Brickwork + Plaster or render on BOTH sides

Permanent action

$$G_2 = \mathbf{2.45 \text{ kN/m}^2} \times \mathbf{2.8 \text{ m}} = \mathbf{6.86 \text{ kN/m}}$$

Variable action

$$Q_2 = \mathbf{0 \text{ kN/m}^2} \times \mathbf{2.8 \text{ m}} = \mathbf{0 \text{ kN/m}}$$



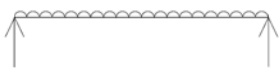
Load 3: UDL - Lightweight timber stud partitions, on floor plan

Permanent action

$$G_3 = \mathbf{0 \text{ kN/m}^2} \times \mathbf{3.5 \text{ m}} = \mathbf{0 \text{ kN/m}}$$

Variable action

$$Q_3 = \mathbf{0.25 \text{ kN/m}^2} \times \mathbf{3.5 \text{ m}} = \mathbf{0.875 \text{ kN/m}}$$



Load 4: UDL - Timber floor (domestic dwelling)

Permanent action

$$G_4 = \mathbf{0.6 \text{ kN/m}^2} \times \mathbf{3.5 \text{ m}} = \mathbf{2.1 \text{ kN/m}}$$

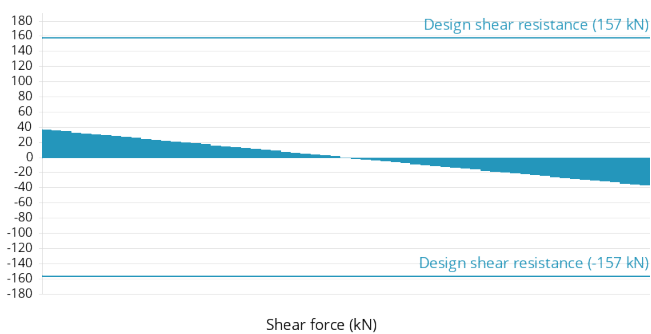
Variable action

$$Q_4 = \mathbf{1.5 \text{ kN/m}^2} \times \mathbf{3.5 \text{ m}} = \mathbf{5.25 \text{ kN/m}}$$

Reactions

| | Permanent (unfactored) | Variable (unfactored) | Total (unfactored) | Total (factored) |
|----------------|------------------------|-----------------------|--------------------|------------------|
| Left reaction | 15.3 kN | 10.5 kN | 25.8 kN | 36.4 kN |
| Right reaction | 15.3 kN | 10.5 kN | 25.8 kN | 36.4 kN |

Design shear force



Design shear force

$$V_{Ed} = \mathbf{36.4 \text{ kN}}$$

Design shear resistance

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

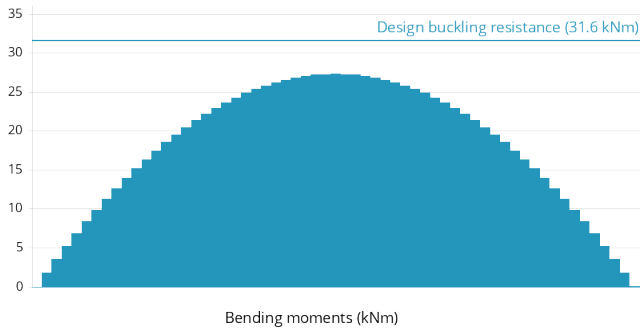
Utilisation

$$V_{Ed} / V_{c,Rd} = \mathbf{23 \%}$$

OK

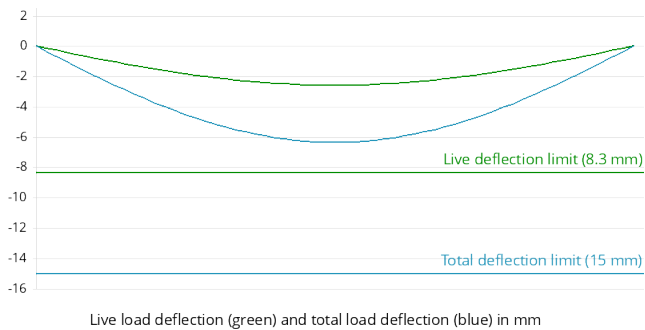
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Design bending moment



| | |
|-----------------------------------|--------------------------------------|
| Design bending moment, major axis | $M_{Ed} = 27.3$ kNm |
| Design resistance for bending | $M_{c,Rd} = 47$ kNm |
| Bending utilisation | $M_{Ed} / M_{c,Rd} = 58\%$ OK |
| Design resistance for buckling | $M_{b,Rd} = 31.6$ kNm |
| Buckling utilisation | $M_{Ed} / M_{b,Rd} = 86\%$ OK |

Deflection



| | |
|----------------------------------|-----------------------------------|
| Variable action deflection limit | $\Delta_Q = 8.3$ mm |
| Variable action deflection | $\delta_Q = 2.6$ mm OK |
| Total deflection limit | $\Delta_{G+Q} = 15$ mm |
| Total deflection | $\delta_{G+Q} = 6.4$ 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 |
| Beta factor for buckling reduction factor calculation | $\beta = 0.75$ | cl.6.3.2.3(1) / BS-EN NA |

Yield strength

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|-----------------------------------------------------------------------------|-------------------------------|----------------|
| Nominal yield strength for S275 grade and nominal section thickness 7.90 mm | $f_y = 275$ N/mm ² | Tata blue book |
|-----------------------------------------------------------------------------|-------------------------------|----------------|

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

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|---------------------------------|-------------------------------|----------------------------|
| Epsilon | $\epsilon = \mathbf{0.924}$ | EN 1993-1-1:2005 Table 5.2 |
| Flange ratio for local buckling | $c_f / t_f = \mathbf{5.14}$ | |
| Flange ratio limit for class 1 | $9 \epsilon = \mathbf{8.32}$ | Table 5.2 (sheet 2 of 3) |
| Flange class | Class _f = 1 | |
| Web ratio for local buckling | $c_w / t_w = \mathbf{30.6}$ | |
| Web ratio limit for class 1 | $72 \epsilon = \mathbf{66.6}$ | Table 5.2 (sheet 1 of 3) |
| Web class | Class _w = 1 | |
| Section class | Class = 1 | |

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

| | | |
|---------------------------------|--------------------------------------|--------------|
| Height of web | $h_w = \mathbf{162}$ mm | |
| Shear area for I and H sections | $A_v = \mathbf{989}$ mm ² | cl.6.2.6 (3) |
| Design shear resistance | $V_{pl,Rd} = \mathbf{157}$ 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 | $h_w / t_w = \mathbf{33.8}$ | EN 1993-1-5:2006 cl.5.1 (2) |
| Shear buckling limit | $72 \epsilon / \eta = \mathbf{66.6}$ | 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 ($157 \text{ kN} / 2 = 79 \text{ kN}$), therefore its effect on moment resistance may be neglected.

| | | |
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| Class 1 section, therefore use plastic modulus | $W_{pl} = \mathbf{171,000}$ mm ³ | |
| Design bending resistance | $M_{c,Rd} = \mathbf{47}$ kNm | eq (6.13) |

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

| | | |
|------------------------------------------------------|---------------------------------------------|----------------------|
| C1 factor | $C1 = \mathbf{1}$ | |
| Shear modulus of elasticity | $G_s = \mathbf{81,000}$ N/mm ² | cl.3.2.6 (1) |
| Buckling length | $L_{cr} = \mathbf{3,000}$ mm | |
| Critical buckling moment | $M_{cR} = \mathbf{43}$ kNm | NCCI SN003b-EN-EU |
| Class 1 section, therefore use plastic modulus | $W_{pl} = \mathbf{171,000}$ mm ³ | cl.6.3.2.1(3) |
| Non-dimensional slenderness ratio | $\bar{\lambda}_{LT} = \mathbf{1.05}$ | cl.6.3.2.2 (1) |
| Depth to width ratio for buckling curve | $h / b = \mathbf{1.76}$ | |
| Buckling curve for h / b ratio | Buckling curve = b | Table 6.5 / BS-EN NA |
| Imperfection factor for buckling curve b | $\alpha_{LT} = \mathbf{0.34}$ | Table 6.3 / BS-EN NA |
| Intermediate factor for reduction factor calculation | $\Phi_{LT} = \mathbf{1.02}$ | cl.6.3.2.3 (1) |
| Buckling reduction factor | $\chi_{LT} = \mathbf{0.671}$ | 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 | $\chi_{LT,mod} = \mathbf{0.671}$ | eq (6.58) |
| Design buckling resistance | $M_{b,Rd} = \mathbf{31.6}$ kNm | eq (6.55) |

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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)