

 structural engineering consultants	Project				Job Ref.	
	8 GORT WAY, HEYWOOD, OL10 1FT				HC-22387	
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1 – INTRODUCTION

The following document is associated with the construction work to take place at the above mentioned address and contains design calculations for structural elements, as well as approximate schematic arrangements of those elements.

2 - DEFINITIONS

The "Engineer" is HEALEY CONSULTING

The "Client" is the individual or organisation that has instructed the engineer to carry out structural engineering consultancy work.

The "Architect" is the individual or organisation that has provided the information upon which these calculations are based.

The "Builder" is the contractor who has been engaged to undertake the construction work to which this document relates.

3 - IMPORTANT GUIDANCE ON THE USE OF THIS DOCUMENT **(TO BE READ BY ALL PARTIES)**

This document is intended to be accompanied by all relevant architects' and engineer's drawings, and all relevant documentation should be considered prior to commencement of the work.

Engineer's drawings relating to this document will be explicitly outlined herein.

This document should be reviewed in its entirety (along with any other relevant documentation) by the builder, architect (if applicable) and client, prior to commencement of the work.

Any layouts, instructions or recommendations should be considered. Any deviations from the proposals outlined herein are to be approved by the Engineer prior to the work being undertaken. Any deviations from the proposals made

without the Engineer's consent are beyond the scope of this document and the Engineer cannot be held liable for any adverse consequences of such deviations.

The calculations within this document have been carried out in good faith based on the data provided by the client and/or builder/architect.

Where applicable extracts of the information provided will be included within this document for reference.

It is the responsibility of the architect (where applicable) or client to notify the engineer when changes are made to the information. This will enable the design to be reviewed and, where necessary, changes made.

Approval of these calculations and drawings by the Local Authority Building Control or similar approved body should be obtained prior to any ordering of material or fabrication. No liability is accepted for any changes that may be required as a result of work having commenced prior to such an approval.

Where information about the existing arrangements of buildings is not available (e.g. floor / roof span orientations or load-bearing wall arrangements) the Engineer will use their judgement to make assumptions. These (generally conservative) assumptions will be clearly outlined within the document, and should be confirmed by a suitably qualified individual on site. The engineer is then to be notified of any discrepancies prior to commencement of the work as design changes may be necessary.

IF IN DOUBT: ASK!!

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It should be noted that until the existing structure has been exposed it is not always possible to foresee what the existing structural arrangement will be. This may include floor and roof span directions, construction materials and existing beam locations among other aspects. It is therefore advised to remove finishes from areas being modified as early as possible to avoid any delays on site that the untimely discovery of such (previously unknown) information can lead to.

In addition, it is advised to carry out some form of ground investigation as early as possible to avoid any unexpected below ground construction requirements which may cause delays on site e.g. piled foundations, raft foundations etc.

Wherever there is any uncertainty we will make every effort to highlight where this lies, however suitable project planning / management by the contractor / client are essential to the smooth execution of any construction project.

4 - HEALTH & SAFETY

It should be noted that structural work, particularly where this involves the transit and installation of large, heavy structural elements, has the potential to be hazardous. Where possible any specific risks are identified either within these calculations or drawings related to this document.

IF IN DOUBT: ASK!!

5 - PARTY WALL AGREEMENTS

If the building work involves:

- work on an existing wall which is shared with another property, or
- building on the boundary with a neighbouring property, or
- excavating near a neighbouring building,

then the client must find out whether the work falls under the Party Wall Act. If it does, the Act requires the client to notify all affected neighbours of proposed works. Refer to your architect for guidance, or see this website for guidance:- <https://www.gov.uk/guidance/partywall-etc-act-1996-guidance>.

6 - CONSTRUCTION NOTES

GENERAL NOTES

Where design GA drawings are issued alongside calculations, the design GA drawings TAKE PRECEDENCE over the calculations.

- Any SPAN / HEIGHT DIMENSIONS shown in this document are for CALCULATION PURPOSES ONLY and are not to be used as a final dimension for the fabrication / machining of structural elements.
- All dimensions are to be checked on site by the builder / contractor / fabricator prior to commencement of fabrication / machining / construction.

Any discrepancies between the information outlined herein and the dimensions on site are to be reported to the engineer.

- Temporary works are the sole responsibility of the builder / contractor. Temporary works method statements are to be provided to the Engineer by the builder / contractor prior to commencement of the work.

- All parties are assumed to be aware of their responsibilities under the Construction Design and Management (CDM) Regulations 2015.

If you are unsure of this please contact the Engineer.

- HEALEY CONSULTING take no responsibility for any elements outside of the scope of these calculations. For structural elements not covered by this document it is assumed that a design is being prepared / provided by others, if additional calculations / drawings / specifications are required then please contact HEALEY CONSULTING and we can provide a fee for their design.

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- All structural work has to be carried out by a competent builder in accordance with the requirements of The Building Regulations Part A and the recommendations set out in BS8103 Parts 1-3.

SIMPLY SUPPORTED BEAM EXAMPLE

- This document is to be read in conjunction with the architectural drawings / details, and any discrepancies reported to the engineer immediately. Final steel levels and dimensions are to be confirmed by the architectural consultant.
- If HEALEY CONSULTING are not contracted to visit site we take no responsibility for the quality of construction nor its compliance with this document. It is the contractor's responsibility to ensure that all works comply with the drawings, notes and assumptions made within these calculations.
- The client / contractor should be aware of beam, frame and column deflections and their impact on existing or adjacent elements, for example beam deflection on bi-fold doors. Typical beam deflections will be span/200 total (Dead+Live) for beams and height/300 (Wind) for columns.

The following calculations provide anticipated deflections within the relevant sections.

- The client should be aware that where beams are installed within existing masonry structures it is likely that minor cracking will occur within the masonry above due to the load redistribution. It is recommended that the contractor allows to make suitable repairs within their contract.
- All proprietary (i.e. off-the-shelf) items specified within this document are to be installed in strict accordance with the manufacturer's recommendations. This includes, but is not limited to restraint straps, lintels, chemical / resin anchors and fixing brackets.
- For structural elements not considered within the scope of this document it is assumed that a suitable specification is to be made by others. If additional calculations, specifications or drawings are required then please contact HEALEY CONSULTING and we can provide a fee for their design.
- At locations where beam bearing information is provided on the layout this will be in a position where load-bearing masonry (with foundations / support) has been assumed. It should be confirmed by a suitably qualified individual that these walls are load-bearing, and the masonry is to be inspected for suitability prior to commencement of the work.

7. DESIGN PHILOSOPHY

All structural members are designed to be capable of withstanding applied loads during construction, operation and maintenance of the building without any distress, failure, loss of function, damage or durability problems. They are to support the most onerous combinations of dead, imposed and (where applicable) wind loads tending to produce either maximum ultimate stresses or deflection.

8. STABILITY STATEMENT

For this project the stability is to be provided by the following structural systems or a combination thereof:

Global Lateral Stability: moment-transmitting frames / masonry piers or shear walls with dead load restoring moment / floors or ceilings to provide diaphragm (boarded & strapped where critical)

Wall Panel Local Lateral Stability: steel or timber posts / masonry piers or walls / floors, ceilings or rafters (boarded & strapped where critical) / timber stud walls.

Vertical Loads: masonry walls, piers or columns with padstones for concentrated loads where necessary / steel posts / strip footings / pad foundations

Internal walls are to be removed. If required sway frames will be introduced

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DESIGN PHILOSOPHY

These calculations should be read in conjunction with drawings **HC-22387-(60)-01** and **Arch's drawings**.

STABILITY STATEMENT

Not Applicable

GROUND BEARING PRESSURES

Not Applicable

Notes:

Timber designs carried out in accordance with BS 5268: Part 2:1996
Masonry designs carried out in accordance with BS 5628: Part 1:1992
Steel beams designed in accordance with BS 5950: Part 1 2000
All beam-ends are to be bolted down to concrete pad stones using
M12 – RAWL R-HPT THROUGH BOLTS ; 100mm Long

LOADING

LOADINGS

	kn/m ²	kn/m ²
<u>ROOF</u>		
Dead + Services + Pbrd Ceiling	1.00	
Live		1.00

TIMBER FLOOR

Dead	
Finishes + Boards	0.15
Insulation	0.05
Timber Joists	0.20
Single boarded ceiling + Plaster	0.15
	<u>0.55</u>

Live

Domestic	1.50
Partitions	0.50
	<u>2.00</u>

WALL

DL	103 brickwork	2.20
	100 blockwork	2.00
	Plaster per face	0.20

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ROOF JOISTS

TIMBER JOIST DESIGN (BS5268)

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

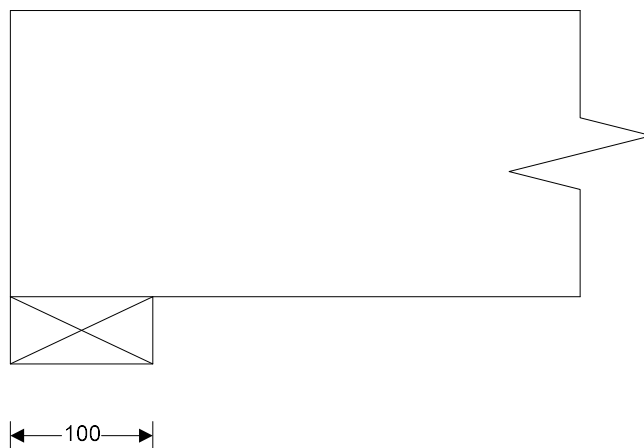
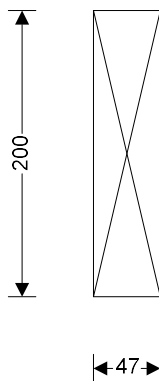
Joist details

Joist breadth; $b = 47 \text{ mm}$
 Joist depth; $h = 200 \text{ mm}$
 Joist spacing; $s = 450 \text{ mm}$
 Timber strength class; **C16**
 Service class of timber; **1**



Span details

Number of spans; $N_{\text{span}} = 1$
 Length of bearing; $L_b = 100 \text{ mm}$
 Effective length of span; $L_{s1} = 3600 \text{ mm}$



Section properties

Second moment of area; $I = b \times h^3 / 12 = 31333333 \text{ mm}^4$
 Section modulus; $Z = b \times h^2 / 6 = 313333 \text{ mm}^3$

Loading details

Joist self weight; $F_{\text{swt}} = b \times h \times \rho_{\text{char}} \times g_{\text{acc}} = 0.03 \text{ kN/m}$
 Dead load; $F_{d_udl} = 0.93 \text{ kN/m}^2$

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Imposed UDL(Long term); $F_{i_udl} = 0.75 \text{ kN/m}^2$
Imposed point load (Medium term); $F_{i_pt} = 1.40 \text{ kN}$

Modification factors

Service class for bending parallel to grain $K_{2m} = 1.00$
Service class for compression $K_{2c} = 1.00$
Service class for shear parallel to grain $K_{2s} = 1.00$
Service class for modulus of elasticity $K_{2e} = 1.00$
Section depth factor; $K_7 = 1.05$
Load sharing factor; $K_8 = 1.10$

Consider long term loads

Load duration factor; $K_3 = 1.00$
;;Maximum bending moment; $M = 1.271 \text{ kNm}$
;;Maximum shear force; $V = 1.412 \text{ kN}$
;;Maximum support reaction; $R = 1.412 \text{ kN}$
;;Maximum deflection; $\delta = 6.518 \text{ mm}$

Check bending stress

Bending stress; $\sigma_m = 5.300 \text{ N/mm}^2$
Permissible bending stress; $\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 6.096 \text{ N/mm}^2$
Applied bending stress; $\sigma_{m_max} = M / Z = 4.056 \text{ N/mm}^2$

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; $\tau = 0.670 \text{ N/mm}^2$
Permissible shear stress; $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.737 \text{ N/mm}^2$
Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h) = 0.225 \text{ N/mm}^2$

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane); $\sigma_{cp1} = 2.200 \text{ N/mm}^2$
Permissible bearing stress; $\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 2.420 \text{ N/mm}^2$
Applied bearing stress; $\sigma_{c_max} = R / (b \times L_b) = 0.300 \text{ N/mm}^2$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 10.800 \text{ mm}$
Bending deflection (based on E_{mean}); $\delta_{bending} = 6.223 \text{ mm}$
Shear deflection; $\delta_{shear} = 0.295 \text{ mm}$
Total deflection; $\delta = \delta_{bending} + \delta_{shear} = 6.518 \text{ mm}$

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor; $K_3 = 1.25$
;;Maximum bending moment; $M = 1.984 \text{ kNm}$
;;Maximum shear force; $V = 2.205 \text{ kN}$
;;Maximum support reaction; $R = 2.205 \text{ kN}$
;;Maximum deflection; $\delta = 8.942 \text{ mm}$

Check bending stress

Bending stress; $\sigma_m = 5.300 \text{ N/mm}^2$
Permissible bending stress; $\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 7.620 \text{ N/mm}^2$

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Applied bending stress;

$$\sigma_{m_max} = M / Z = 6.333 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress;

$$\tau = 0.670 \text{ N/mm}^2$$

Permissible shear stress;

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

Applied shear stress;

$$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.352 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane);

$$\sigma_{cp1} = 2.200 \text{ N/mm}^2$$

Permissible bearing stress;

$$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 3.025 \text{ N/mm}^2$$

Applied bearing stress;

$$\sigma_{c_max} = R / (b \times L_b) = 0.469 \text{ N/mm}^2$$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection;

$$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 10.800 \text{ mm}$$

Bending deflection (based on E_{mean});

$$\delta_{bending} = 8.481 \text{ mm}$$

Shear deflection;

$$\delta_{shear} = 0.461 \text{ mm}$$

Total deflection;

$$\delta = \delta_{bending} + \delta_{shear} = 8.942 \text{ mm}$$

PASS - Actual deflection within permissible limits

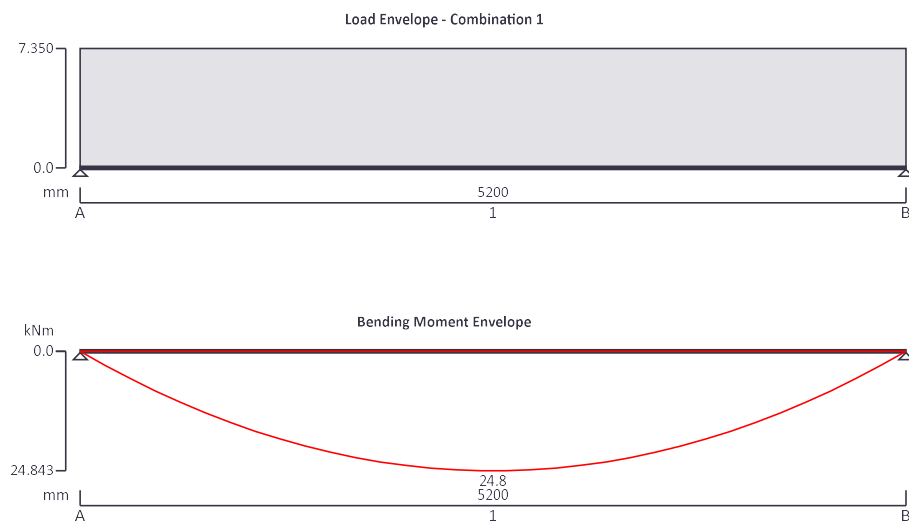
RIDGE BEAM

STEEL BEAM ANALYSIS & DESIGN (BS5950)

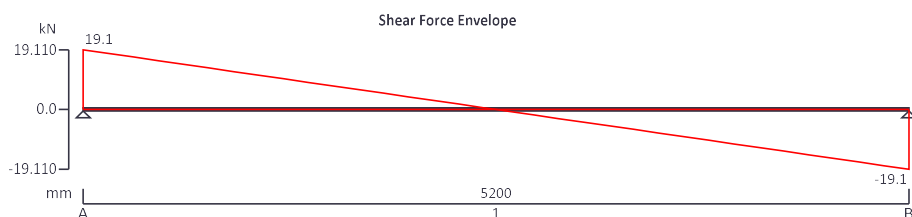
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07



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Support conditions

Support A	Vertically restrained Rotationally free
Support B	Vertically restrained Rotationally free

Applied loading

Beam loads	Dead self weight of beam $\times 1$ Roof - Dead full UDL 2.604 kN/m Roof - Imposed full UDL 2.1 kN/m
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Load combinations

Load combination 1	Support A	Dead $\times 1.40$ Imposed $\times 1.60$ Dead $\times 1.40$ Imposed $\times 1.60$
	Support B	Dead $\times 1.40$ Imposed $\times 1.60$

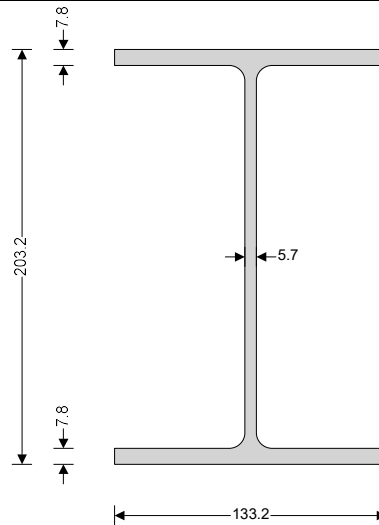
Analysis results

Maximum moment;	$M_{\max} = 24.8 \text{ kNm};$	$M_{\min} = 0 \text{ kNm}$
Maximum shear;	$V_{\max} = 19.1 \text{ kN};$	$V_{\min} = -19.1 \text{ kN}$
Deflection;	$\delta_{\max} = 9.8 \text{ mm};$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A;	$R_{A_{\max}} = 19.1 \text{ kN};$	$R_{A_{\min}} = 19.1 \text{ kN}$
Unfactored dead load reaction at support A;	$R_{A_{\text{Dead}}} = 7.4 \text{ kN}$	
Unfactored imposed load reaction at support A;	$R_{A_{\text{Imposed}}} = 5.5 \text{ kN}$	
Maximum reaction at support B;	$R_{B_{\max}} = 19.1 \text{ kN};$	$R_{B_{\min}} = 19.1 \text{ kN}$
Unfactored dead load reaction at support B;	$R_{B_{\text{Dead}}} = 7.4 \text{ kN}$	
Unfactored imposed load reaction at support B;	$R_{B_{\text{Imposed}}} = 5.5 \text{ kN}$	

Section details

Section type;	UB 203x133x25 (BS4-1)
Steel grade;	S275
From table 9: Design strength p_y	
Thickness of element;	$\max(T, t) = 7.8 \text{ mm}$
Design strength;	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity;	$E = 205000 \text{ N/mm}^2$

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Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis;

$$K_x = 1.00$$

Effective length factor in minor axis;

$$K_y = 1.00$$

Effective length factor for lateral-torsional buckling;

$$K_{LTA} = 1.00; + 2 \times D$$

$$K_{LTB} = 1.00; + 2 \times D$$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$$

Internal compression parts - Table 11

Depth of section;

$$d = 172.4 \text{ mm}$$

$$d / t = 30.2 \times \varepsilon \leq 80 \times \varepsilon; \quad \text{Class 1 plastic}$$

Outstand flanges - Table 11

Width of section;

$$b = B / 2 = 66.6 \text{ mm}$$

$$b / T = 8.5 \times \varepsilon \leq 9 \times \varepsilon; \quad \text{Class 1 plastic}$$

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force;

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 19.1 \text{ kN}$$

$$d / t < 70 \times \varepsilon$$

Web does not need to be checked for shear buckling

Shear area;

$$A_v = t \times D = 1158 \text{ mm}^2$$

Design shear resistance;

$$P_v = 0.6 \times p_y \times A_v = 191.1 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment;

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 24.8 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2;

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 70.9 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling;

$$L_E = 1.0 \times L_{s1} + 2 \times D = 5606 \text{ mm}$$

Slenderness ratio;

$$\lambda = L_E / r_{yy} = 180.730$$

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Equivalent slenderness - Section 4.3.6.7

Buckling parameter;	$u = 0.877$
Torsional index;	$x = 25.605$
Slenderness factor;	$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.732$
Ratio - cl.4.3.6.9;	$\beta_W = 1.000$
Equivalent slenderness - cl.4.3.6.7;	$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_W} = 115.949$
Limiting slenderness - Annex B.2.2;	$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$
	$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant;	$\alpha_{LT} = 7.0$
Perry factor;	$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.571$
Euler stress;	$p_E = \pi^2 \times E / \lambda_{LT}^2 = 150.5 \text{ N/mm}^2$
	$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 255.7 \text{ N/mm}^2$
Bending strength - Annex B.2.1;	$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 100.8 \text{ N/mm}^2$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment;	$M_2 = 18.6 \text{ kNm}$
Moment at centre-line of segment;	$M_3 = 24.8 \text{ kNm}$
Moment at three quarter point of segment;	$M_4 = 18.6 \text{ kNm}$
Maximum moment in segment;	$M_{abs} = 24.8 \text{ kNm}$
Maximum moment governing buckling resistance;	$M_{LT} = M_{abs} = 24.8 \text{ kNm}$
Equivalent uniform moment factor for lateral-torsional buckling;	$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.925$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment;	$M_b = p_b \times S_{xx} = 26 \text{ kNm}$
	$M_b / m_{LT} = 28.1 \text{ kNm}$
PASS - Buckling resistance moment exceeds design bending moment	

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads	
Limiting deflection;	$\delta_{lim} = L_{s1} / 360 = 14.444 \text{ mm}$
Maximum deflection span 1;	$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 9.823 \text{ mm}$
PASS - Maximum deflection does not exceed deflection limit	

CONCRETE PADSTONE

MASONRY BEARING DESIGN (BS5628)

MASONRY BEARING DESIGN TO BS5628-1:2005

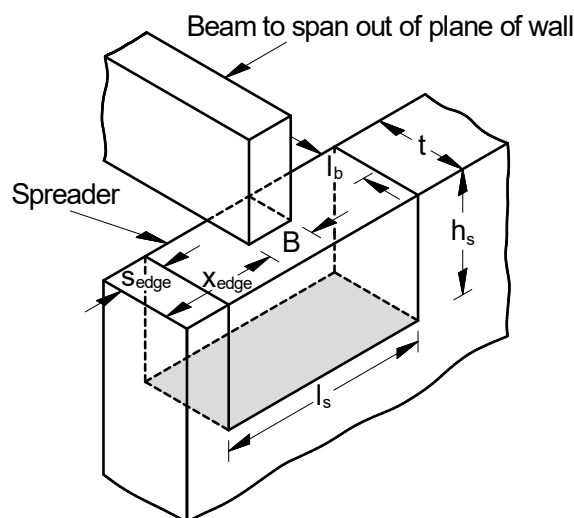
TEDDS calculation version 1.0.06

Masonry details

Masonry type;	Aggregate concrete blocks (25% or less formed voids)
Compressive strength of unit;	$p_{unit} = 3.6 \text{ N/mm}^2$
Mortar designation;	iii
Least horizontal dimension of masonry units;	$l_{unit} = 100 \text{ mm}$
Height of masonry units;	$h_{unit} = 215 \text{ mm}$

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Category of masonry units;	Category II
Category of construction control ;	Normal
Partial safety factor for material strength;	$\gamma_m = 3.5$
Thickness of load bearing leaf;	$t = 100 \text{ mm}$
Effective thickness of masonry wall;	$t_{ef} = 100 \text{ mm}$
Height of masonry wall;	$h = 2400 \text{ mm}$
Effective height of masonry wall;	$h_{ef} = 2400 \text{ mm}$



Bearing details

Beam spanning out of plane of wall

Width of bearing;	$B = 100 \text{ mm}$
Length of bearing;	$l_b = 100 \text{ mm}$
Edge distance;	$X_{edge} = 75 \text{ mm}$

Compressive strength from Table 2 BS5628:Part 1 - aggregate concrete blocks (25% or less formed voids)

;Mortar designation;	Mortar = "iii"
;Block compressive strength;	$p_{unit} = 3.6 \text{ N/mm}^2$
;;;;;Characteristic compressive strength (Table 2c);	$f_{kc} = 1.70 \text{ N/mm}^2$
;;;;;Characteristic compressive strength (Table 2d);	$f_{kd} = 3.50 \text{ N/mm}^2$
;Height of solid block;	$h_{unit} = 215.0 \text{ mm}$;
;Least horizontal dimension;	$l_{unit} = 100.0 \text{ mm}$
Block ratio;	ratio = $h_{unit} / l_{unit} = 2.2$

Ratio between 0.6 and 4.5 - OK

Characteristic compressive strength;	$f_k = 3.50 \text{ N/mm}^2$
--------------------------------------	-----------------------------

Loading details

Characteristic concentrated dead load;	$G_k = 8 \text{ kN}$
Characteristic concentrated imposed load;	$Q_k = 6 \text{ kN}$
Design concentrated load;	$F = (G_k \times 1.4) + (Q_k \times 1.6) = 20.8 \text{ kN}$
Characteristic distributed dead load;	$g_k = 0.0 \text{ kN/m}$
Characteristic distributed imposed load;	$q_k = 0.0 \text{ kN/m}$

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Design distributed load;

$$f = (g_k \times 1.4) + (q_k \times 1.6) = \mathbf{0.0 \text{ kN/m}}$$

Masonry bearing type

Bearing type;

Type 1

Bearing safety factor;

$$\gamma_{\text{bear}} = \mathbf{1.25}$$

Check design bearing without a spreader

Design bearing stress;

$$f_{ca} = F / (B \times l_b) + f / t = \mathbf{2.080 \text{ N/mm}^2}$$

Allowable bearing stress;

$$f_{cp} = \gamma_{\text{bear}} \times f_k / \gamma_m = \mathbf{1.250 \text{ N/mm}^2}$$

;NOTE - Design bearing stress exceeds allowable bearing stress, use a spreader

Spreader details

;Length of spreader;

$$l_s = \mathbf{330 \text{ mm}}$$

Depth of spreader;

$$h_s = \mathbf{215 \text{ mm}}$$

Edge distance;

$$s_{\text{edge}} = \max(0 \text{ mm}, x_{\text{edge}} - (l_s - B) / 2) = \mathbf{0 \text{ mm}}$$

Spreader bearing type

;Bearing type;

Type 3

Bearing safety factor;

$$\gamma_{\text{bear}} = \mathbf{2.00}$$

Check design bearing with a spreader

Loading acts eccentrically within middle third – triangular stress distribution

Eccentricity of load;

$$e = ((l_s - B) / 2) - x_{\text{edge}} = \mathbf{40 \text{ mm}}$$

Maximum bearing stress;

$$f_{ca} = F \times (1 + (6 \times e / l_s)) / (l_s \times t) + f / t = \mathbf{1.089 \text{ N/mm}^2}$$

Allowable bearing stress;

$$f_{cp} = \gamma_{\text{bear}} \times f_k / \gamma_m = \mathbf{2.000 \text{ N/mm}^2}$$

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at $0.4 \times h$ below the bearing level

Slenderness ratio;

$$h_{\text{ef}} / t_{\text{ef}} = \mathbf{24.00}$$

Eccentricity at top of wall;

$$e_x = \mathbf{0.0 \text{ mm}}$$

From BS5628:1 Table 7

Capacity reduction factor;

$$\beta = \mathbf{0.61}$$

Length of bearing distributed at $0.4 \times h$;

$$l_d = \mathbf{1135 \text{ mm}}$$

Maximum bearing stress;

$$f_{ca} = F / (l_d \times t) + f / t = \mathbf{0.183 \text{ N/mm}^2}$$

Allowable bearing stress;

$$f_{cp} = \beta \times f_k / \gamma_m = \mathbf{0.605 \text{ N/mm}^2}$$

PASS - Allowable bearing stress at $0.4 \times h$ below bearing level exceeds design bearing stress

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LOADBEARING STUD WALL

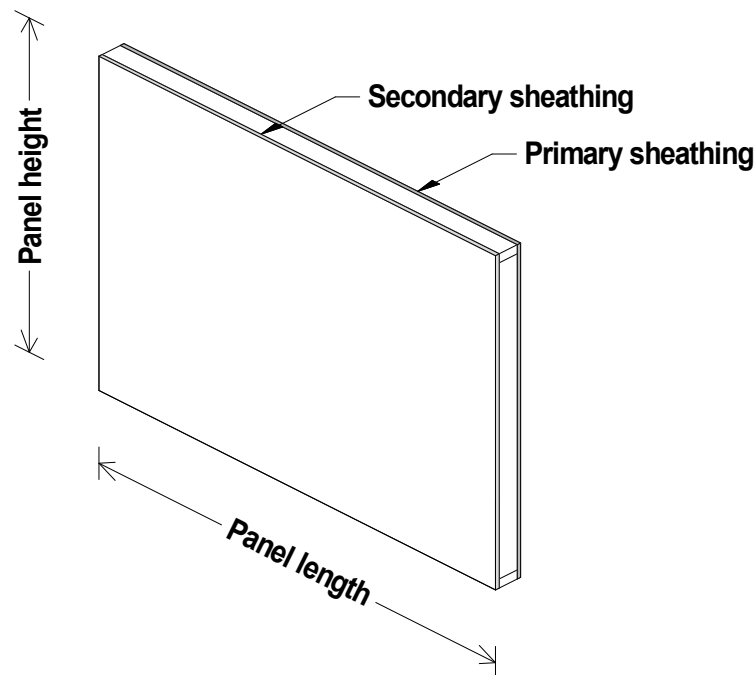
TIMBER FRAME RACKING PANEL DESIGN (BS5268)

TIMBER PANEL RACKING RESISTANCE – BS5268:SECTION 6.1:1996

TEDDS calculation version 1.0.05

Dwellings not exceeding seven storeys

wall1



Wall panel details

Length of panel; $L = 2.400$ m
Height of panel; $H_{wp} = 2.400$ m;
Total area of wall panel; $A_t = L \times H_{wp} = 5.760$ m²
Aggregate area of framed panel openings; $A_a = 0.000$ m²;
Timber members; **38 mm x 72 mm or larger**
Uniformly distributed load on timber frame wall; $F_{udl} = 3.400$ kN/m
For calculation equivalent uniformly distributed load; $F = \min(F_{udl}, 10.5 \text{ kN/m}) = 3.400$ kN/m

Primary sheathing details

Primary board type; **Plasterboard**
Standard board thickness; $t_p = 12.50$ mm
Proposed board thickness; $T_p = 12.50$ mm;
Ratio of proposed to standard board thickness; $B_p = \min(\max(T_p / t_p, 0.75), 1.25) = 1.00$
Screw diameter; $D_p = 3.50$ mm
Standard perimeter screw spacing; $s_p = 150$ mm
Proposed perimeter screw spacing; $S_p = 300$ mm

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From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance; $R_{bp} = 0.400 \text{ kN/m}$;

Modification factors for variation in fixing and thickness of primary sheathing

Variation in screw diameter; $K_{101p} = 1.000$

Variation in screw spacing; $K_{102p} = 1.000$

Variation in board thickness; $K_{103p} = 2.8 \times B_p - B_p^2 - 0.8 = 1.000$

Material modification factors; $K_{mp} = K_{101p} \times K_{102p} \times K_{103p} = 1.000$

Secondary sheathing details

Secondary board type; **Plasterboard**

Standard board thickness; $t_s = 12.50 \text{ mm}$

Proposed board thickness; $T_s = 12.50 \text{ mm}$;

Ratio of proposed to standard board thickness; $B_s = \min(\max(T_s / t_s, 0.75), 1.25) = 1.00$

Screw diameter; $D_s = 3.50 \text{ mm}$

Standard perimeter screw spacing; $s_s = 150 \text{ mm}$

Proposed perimeter screw spacing; $S_s = 300 \text{ mm}$

From Table 2 – Basic racking resistance for a range of materials and combinations of materials

Basic racking resistance; $R_{bs} = 0.200 \text{ kN/m}$;

Modification factors for variation in fixing and thickness of secondary sheathing

Variation in screw diameter; $K_{101s} = 1.000$

Variation in screw spacing; $K_{102s} = 1.000$

Variation in board thickness; $K_{103s} = 2.8 \times B_s - B_s^2 - 0.8 = 1.000$

Material modification factors; $K_{ms} = K_{101s} \times K_{102s} \times K_{103s} = 1.000$

Modification factors for wall height, length, openings, vertical load and interaction

Height of wall panels; $K_{104} = 2.4 \text{ m} / H_{wp} = 1.000$

Length of walls; $K_{105} = L / 2.4 \text{ m} = 1.000$

Fully framed openings in walls; $K_{106} = (1 - 1.3 \times A_a / A_t)^2 = 1.000$

Vertical load on timber frame wall; $K_{107} = 1 + [(0.09 \times (F / 1 \text{ kN/m}) - 0.0015 \times (F / 1 \text{ kN/m})^2) \times (2.4 \text{ m} / L)^{0.4}]$
 $K_{107} = 1.289$

Interaction; $K_{108} = 1.100$

Wall modification factors; $K_w = K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108} = 1.418$

Racking resistance of wall panel

Racking resistance of wall panel; $R_R = L \times K_w \times (R_{bp} \times K_{mp} + R_{bs} \times K_{ms}) = 2.041 \text{ kN}$

Racking resistance of plasterboard only; $R_{PO} = L \times K_w \times (R_{bp} \times K_{mp} + R_{bs} \times K_{ms}) = 2.041 \text{ kN}$

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FLOOR JOISTS (LOFT)

TIMBER JOIST DESIGN (BS5268)

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

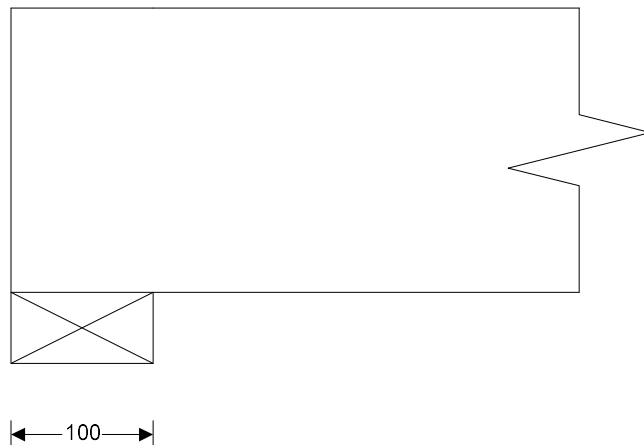
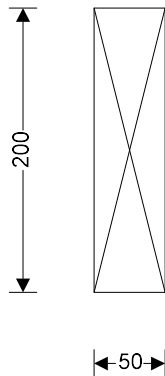
Joist details

Joist breadth;	b = 50 mm
Joist depth;	h = 200 mm
Joist spacing;	s = 400 mm
Timber strength class;	C16
Service class of timber;	1



Span details

Number of spans;	N_{span} = 1
Length of bearing;	L_b = 100 mm
Effective length of span;	L_{s1} = 3750 mm



Section properties

Second moment of area;	I = b × h³ / 12 = 33333333 mm⁴
Section modulus;	Z = b × h² / 6 = 333333 mm³

Loading details

Joist self weight;	F_{swt} = b × h × ρ_{char} × g_{acc} = 0.03 kN/m
Dead load;	F_{d_udl} = 0.60 kN/m²
Imposed UDL(Long term);	F_{i_udl} = 2.00 kN/m²
Imposed point load (Medium term);	F_{i_pt} = 1.40 kN

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Modification factors

Service class for bending parallel to grain	$K_{2m} = 1.00$
Service class for compression	$K_{2c} = 1.00$
Service class for shear parallel to grain	$K_{2s} = 1.00$
Service class for modulus of elasticity	$K_{2e} = 1.00$
Section depth factor;	$K_7 = 1.05$
Load sharing factor;	$K_8 = 1.10$

Consider long term loads

Load duration factor;	$K_3 = 1.00$
;;Maximum bending moment;	$M = 1.882 \text{ kNm}$
;;Maximum shear force;	$V = 2.007 \text{ kN}$
;;Maximum support reaction;	$R = 2.007 \text{ kN}$
;;Maximum deflection;	$\delta = 9.807 \text{ mm}$

Check bending stress

Bending stress;	$\sigma_m = 5.300 \text{ N/mm}^2$
Permissible bending stress;	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 6.096 \text{ N/mm}^2$
Applied bending stress;	$\sigma_{m_max} = M / Z = 5.645 \text{ N/mm}^2$
PASS - Applied bending stress within permissible limits	

Check shear stress

Shear stress;	$\tau = 0.670 \text{ N/mm}^2$
Permissible shear stress;	$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.737 \text{ N/mm}^2$
Applied shear stress;	$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.301 \text{ N/mm}^2$
PASS - Applied shear stress within permissible limits	

Check bearing stress

Compression perpendicular to grain (no wane);	$\sigma_{cp1} = 2.200 \text{ N/mm}^2$
Permissible bearing stress;	$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 2.420 \text{ N/mm}^2$
Applied bearing stress;	$\sigma_{c_max} = R / (b \times L_b) = 0.401 \text{ N/mm}^2$
PASS - Applied bearing stress within permissible limits	

Check deflection

Permissible deflection;	$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 11.250 \text{ mm}$
Bending deflection (based on E_{mean});	$\delta_{bending} = 9.396 \text{ mm}$
Shear deflection;	$\delta_{shear} = 0.411 \text{ mm}$
Total deflection;	$\delta = \delta_{bending} + \delta_{shear} = 9.807 \text{ mm}$
PASS - Actual deflection within permissible limits	

Consider medium term loads

Load duration factor;	$K_3 = 1.25$
;;Maximum bending moment;	$M = 1.788 \text{ kNm}$
;;Maximum shear force;	$V = 1.907 \text{ kN}$
;;Maximum support reaction;	$R = 1.907 \text{ kN}$
;;Maximum deflection;	$\delta = 8.007 \text{ mm}$

Check bending stress

Bending stress;	$\sigma_m = 5.300 \text{ N/mm}^2$
Permissible bending stress;	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 7.620 \text{ N/mm}^2$
Applied bending stress;	$\sigma_{m_max} = M / Z = 5.363 \text{ N/mm}^2$

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PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress;

$$\tau = 0.670 \text{ N/mm}^2$$

Permissible shear stress;

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

Applied shear stress;

$$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.286 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane);

$$\sigma_{cp1} = 2.200 \text{ N/mm}^2$$

Permissible bearing stress;

$$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 3.025 \text{ N/mm}^2$$

Applied bearing stress;

$$\sigma_{c_max} = R / (b \times L_b) = 0.381 \text{ N/mm}^2$$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection;

$$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 11.250 \text{ mm}$$

Bending deflection (based on E_{mean});

$$\delta_{bending} = 7.617 \text{ mm}$$

Shear deflection;

$$\delta_{shear} = 0.390 \text{ mm}$$

Total deflection;

$$\delta = \delta_{bending} + \delta_{shear} = 8.007 \text{ mm}$$

PASS - Actual deflection within permissible limits

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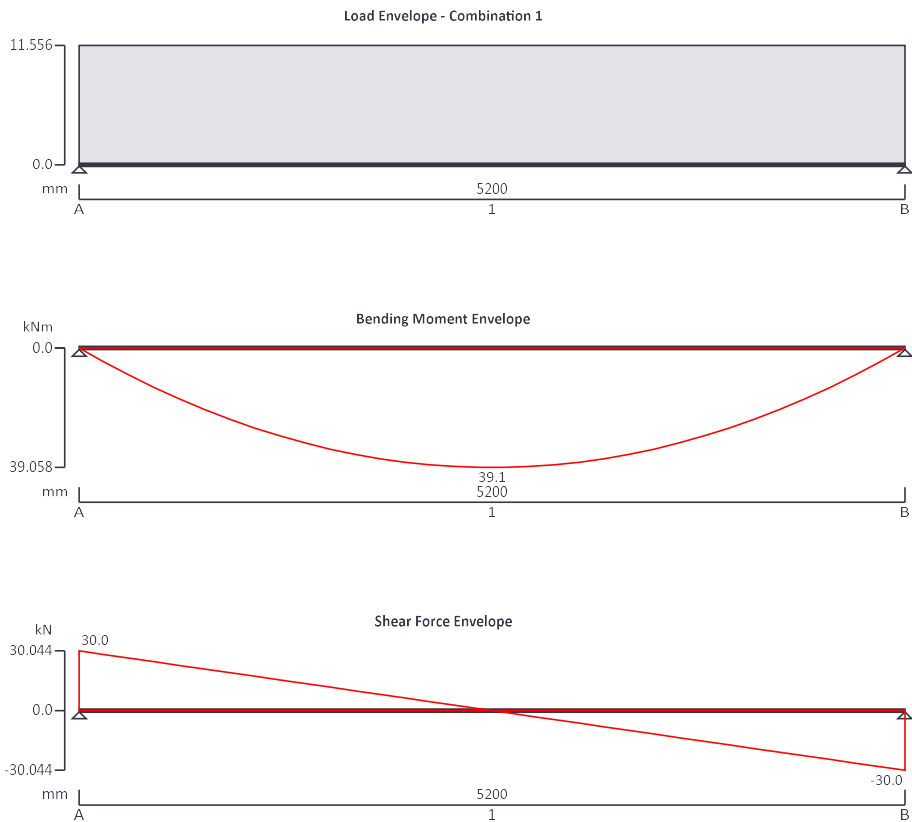
BEAM B1

STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times$ 1

Floor - Dead full UDL 1.665 kN/m

Floor - Imposed full UDL 5.55 kN/m

Load combinations

Load combination 1

Support A

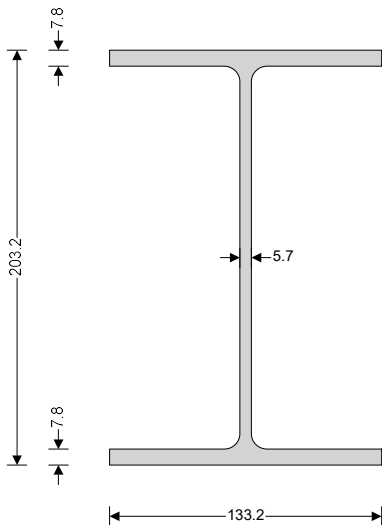
Dead $\times$ 1.40

Imposed $\times$ 1.60

Dead $\times$ 1.40

Imposed $\times$ 1.60

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Support B		Dead $\times 1.40$
		Imposed $\times 1.60$
Analysis results		
Maximum moment;	$M_{\max} = 39.1 \text{ kNm};$	$M_{\min} = 0 \text{ kNm}$
Maximum shear;	$V_{\max} = 30 \text{ kN};$	$V_{\min} = -30 \text{ kN}$
Deflection;	$\delta_{\max} = 11 \text{ mm};$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A;	$R_{A_{\max}} = 30 \text{ kN};$	$R_{A_{\min}} = 30 \text{ kN}$
Unfactored dead load reaction at support A;	$R_{A_{\text{Dead}}} = 5 \text{ kN}$	
Unfactored imposed load reaction at support A;	$R_{A_{\text{Imposed}}} = 14.4 \text{ kN}$	
Maximum reaction at support B;	$R_{B_{\max}} = 30 \text{ kN};$	$R_{B_{\min}} = 30 \text{ kN}$
Unfactored dead load reaction at support B;	$R_{B_{\text{Dead}}} = 5 \text{ kN}$	
Unfactored imposed load reaction at support B;	$R_{B_{\text{Imposed}}} = 14.4 \text{ kN}$	
Section details		
Section type;	UB 203x133x25 (BS4-1)	
Steel grade;	S275	
From table 9: Design strength p_y		
Thickness of element;	$\max(T, t) = 7.8 \text{ mm}$	
Design strength;	$p_y = 275 \text{ N/mm}^2$	
Modulus of elasticity;	$E = 205000 \text{ N/mm}^2$	
		
Lateral restraint		
	Span 1 has full lateral restraint	
Effective length factors		
Effective length factor in major axis;	$K_x = 1.00$	
Effective length factor in minor axis;	$K_y = 1.00$	
Effective length factor for lateral-torsional buckling;	$K_{LT,A} = 1.00; + 2 \times D$	
	$K_{LT,B} = 1.00; + 2 \times D$	
Classification of cross sections - Section 3.5		
	$\varepsilon = \sqrt{275 \text{ N/mm}^2 / p_y} = 1.00$	
Internal compression parts - Table 11		
Depth of section;	$d = 172.4 \text{ mm}$	
	$d / t = 30.2 \times \varepsilon \leq 80 \times \varepsilon;$	Class 1 plastic

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Outstand flanges - Table 11

Width of section;

$$b = B / 2 = 66.6 \text{ mm}$$

$$b / T = 8.5 \times \varepsilon \leq 9 \times \varepsilon;$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force;

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 30 \text{ kN}$$

$$d / t < 70 \times \varepsilon$$

Web does not need to be checked for shear buckling

Shear area;

$$A_v = t \times D = 1158 \text{ mm}^2$$

Design shear resistance;

$$P_v = 0.6 \times p_y \times A_v = 191.1 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment;

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 39.1 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2;

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 70.9 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection;

$$\delta_{lim} = L_{s1} / 360 = 14.444 \text{ mm}$$

Maximum deflection span 1;

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 11.014 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

CONCRETE PADSTONE

MASONRY BEARING DESIGN (BS5628)

MASONRY BEARING DESIGN TO BS5628-1:2005

TEDDS calculation version 1.0.06

Masonry details

Masonry type;

Compressive strength of unit;

Mortar designation;

Least horizontal dimension of masonry units;

Height of masonry units;

Category of masonry units;

Category of construction control ;

Partial safety factor for material strength;

Thickness of load bearing leaf;

Effective thickness of masonry wall;

Height of masonry wall;

Effective height of masonry wall;

Aggregate concrete blocks (25% or less formed voids)

$$p_{unit} = 3.6 \text{ N/mm}^2$$

iii

$$l_{unit} = 100 \text{ mm}$$

$$h_{unit} = 215 \text{ mm}$$

Category II

Normal

$$\gamma_m = 3.5$$

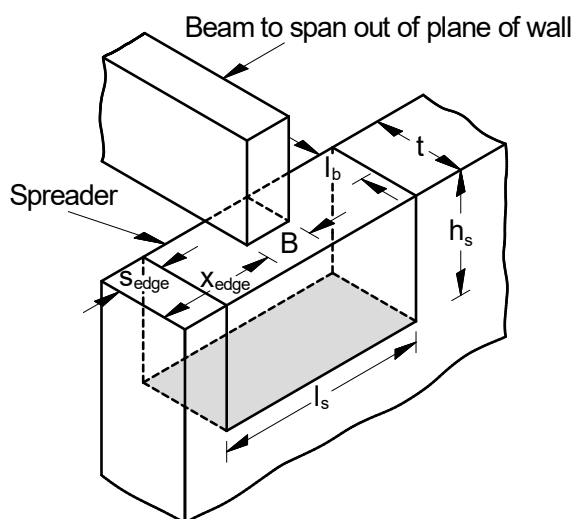
$$t = 100 \text{ mm}$$

$$t_{ef} = 100 \text{ mm}$$

$$h = 2400 \text{ mm}$$

$$h_{ef} = 2400 \text{ mm}$$

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Bearing details

Beam spanning out of plane of wall

Width of bearing; $B = 100 \text{ mm}$
Length of bearing; $l_b = 100 \text{ mm}$
Edge distance; $X_{\text{edge}} = 75 \text{ mm}$

Compressive strength from Table 2 BS5628:Part 1 - aggregate concrete blocks (25% or less formed voids)

Mortar designation; Mortar = "iii"
Block compressive strength; $p_{\text{unit}} = 3.6 \text{ N/mm}^2$
Characteristic compressive strength (Table 2c); $f_{kc} = 1.70 \text{ N/mm}^2$
Characteristic compressive strength (Table 2d); $f_{kd} = 3.50 \text{ N/mm}^2$
Height of solid block; $h_{\text{unit}} = 215.0 \text{ mm}$
Least horizontal dimension; $l_{\text{unit}} = 100.0 \text{ mm}$
Block ratio; $\text{ratio} = h_{\text{unit}} / l_{\text{unit}} = 2.2$

Ratio between 0.6 and 4.5 - OK

Characteristic compressive strength; $f_k = 3.50 \text{ N/mm}^2$

Loading details

Characteristic concentrated dead load; $G_k = 5 \text{ kN}$
Characteristic concentrated imposed load; $Q_k = 15 \text{ kN}$
Design concentrated load; $F = (G_k \times 1.4) + (Q_k \times 1.6) = 31.0 \text{ kN}$
Characteristic distributed dead load; $g_k = 0.0 \text{ kN/m}$
Characteristic distributed imposed load; $q_k = 0.0 \text{ kN/m}$
Design distributed load; $f = (g_k \times 1.4) + (q_k \times 1.6) = 0.0 \text{ kN/m}$

Masonry bearing type

Bearing type; **Type 1**
Bearing safety factor; $\gamma_{\text{bear}} = 1.25$

Check design bearing without a spreader

Design bearing stress; $f_{ca} = F / (B \times l_b) + f / t = 3.100 \text{ N/mm}^2$

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Allowable bearing stress; $f_{cp} = \gamma_{bear} \times f_k / \gamma_m = 1.250 \text{ N/mm}^2$

;NOTE - Design bearing stress exceeds allowable bearing stress, use a spreader

Spreader details

;Length of spreader; $l_s = 330 \text{ mm}$

Depth of spreader; $h_s = 215 \text{ mm}$

Edge distance; $s_{edge} = \max(0 \text{ mm}, x_{edge} - (l_s - B) / 2) = 0 \text{ mm}$

Spreader bearing type

;Bearing type; **Type 3**

Bearing safety factor; $\gamma_{bear} = 2.00$

Check design bearing with a spreader

Loading acts eccentrically within middle third – triangular stress distribution

Eccentricity of load; $e = ((l_s - B) / 2) - x_{edge} = 40 \text{ mm}$

Maximum bearing stress; $f_{ca} = F \times (1 + (6 \times e / l_s)) / (l_s \times t) + f / t = 1.623 \text{ N/mm}^2$

Allowable bearing stress; $f_{cp} = \gamma_{bear} \times f_k / \gamma_m = 2.000 \text{ N/mm}^2$

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at $0.4 \times h$ below the bearing level

Slenderness ratio; $h_{ef} / t_{ef} = 24.00$

Eccentricity at top of wall; $e_x = 0.0 \text{ mm}$

From BS5628:1 Table 7

Capacity reduction factor; $\beta = 0.61$

Length of bearing distributed at $0.4 \times h$; $l_d = 1135 \text{ mm}$

Maximum bearing stress; $f_{ca} = F / (l_d \times t) + f / t = 0.273 \text{ N/mm}^2$

Allowable bearing stress; $f_{cp} = \beta \times f_k / \gamma_m = 0.605 \text{ N/mm}^2$

PASS - Allowable bearing stress at $0.4 \times h$ below bearing level exceeds design bearing stress

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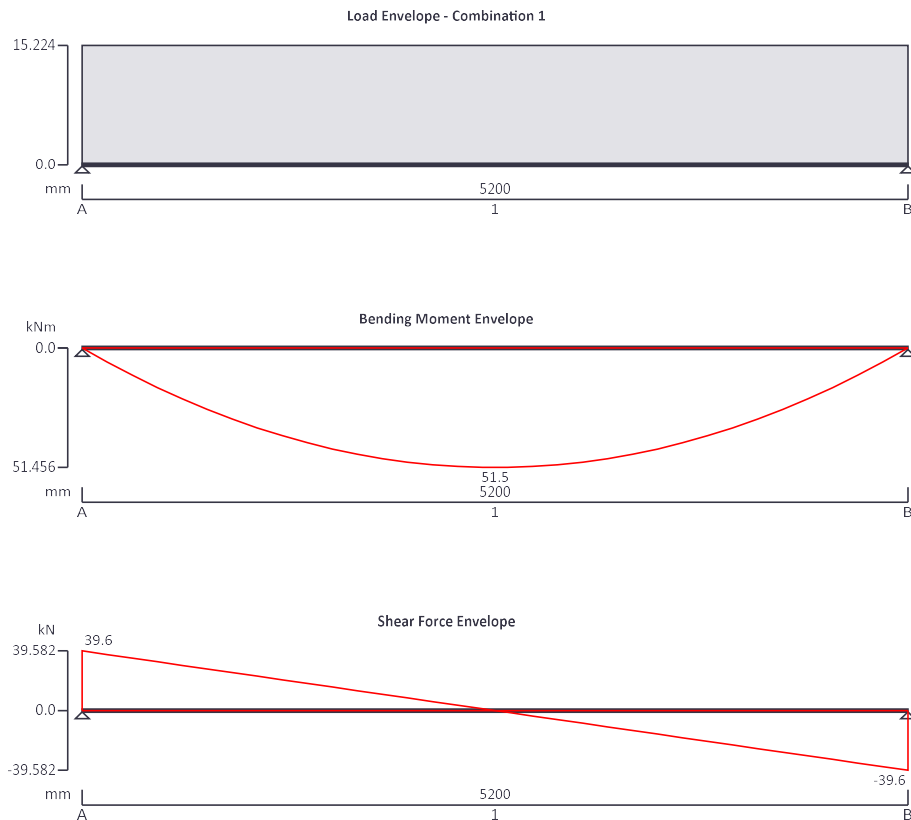
BEAM B2

STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Floor - Dead full UDL 1.125 kN/m

Floor - Imposed full UDL 3.75 kN/m

Roof - Dead full UDL 1.82 kN/m

Roof - Imposed full UDL 1.47 kN/m

Stud wall - Dead full UDL 1.3 kN/m

Ceiling - Dead full UDL 0.143 kN/m

Ceiling - Imposed full UDL 0.24 kN/m

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Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

Imposed $\times 1.60$

Analysis results

Maximum moment;

$M_{\max} = 51.5 \text{ kNm}$;

$M_{\min} = 0 \text{ kNm}$

Maximum shear;

$V_{\max} = 39.6 \text{ kN}$;

$V_{\min} = -39.6 \text{ kN}$

Deflection;

$\delta_{\max} = 10.8 \text{ mm}$;

$\delta_{\min} = 0 \text{ mm}$

Maximum reaction at support A;

$R_{A_{\max}} = 39.6 \text{ kN}$;

$R_{A_{\min}} = 39.6 \text{ kN}$

Unfactored dead load reaction at support A;

$R_{A_{\text{Dead}}} = 12 \text{ kN}$

Unfactored imposed load reaction at support A;

$R_{A_{\text{Imposed}}} = 14.2 \text{ kN}$

Maximum reaction at support B;

$R_{B_{\max}} = 39.6 \text{ kN}$;

$R_{B_{\min}} = 39.6 \text{ kN}$

Unfactored dead load reaction at support B;

$R_{B_{\text{Dead}}} = 12 \text{ kN}$

Unfactored imposed load reaction at support B;

$R_{B_{\text{Imposed}}} = 14.2 \text{ kN}$

Section details

Section type;

UB 203x133x25 (BS4-1)

Steel grade;

S275

From table 9: Design strength p_y

Thickness of element;

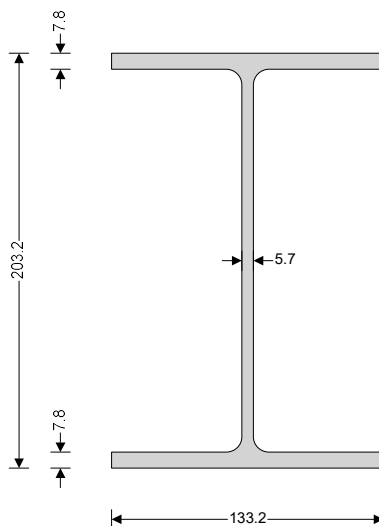
$\max(T, t) = 7.8 \text{ mm}$

Design strength;

$p_y = 275 \text{ N/mm}^2$

Modulus of elasticity;

$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis;

$K_x = 1.00$

Effective length factor in minor axis;

$K_y = 1.00$

Effective length factor for lateral-torsional buckling;

$K_{LTA} = 1.00; + 2 \times D$

$K_{LTB} = 1.20; + 2 \times D$

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Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$$

Internal compression parts - Table 11

Depth of section;

$$d = 172.4 \text{ mm}$$

$$d / t = 30.2 \times \varepsilon \leq 80 \times \varepsilon; \quad \text{Class 1 plastic}$$

Outstand flanges - Table 11

Width of section;

$$b = B / 2 = 66.6 \text{ mm}$$

$$b / T = 8.5 \times \varepsilon \leq 9 \times \varepsilon; \quad \text{Class 1 plastic}$$

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force;

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 39.6 \text{ kN}$$

$$d / t < 70 \times \varepsilon$$

Web does not need to be checked for shear buckling

Shear area;

$$A_v = t \times D = 1158 \text{ mm}^2$$

Design shear resistance;

$$P_v = 0.6 \times p_y \times A_v = 191.1 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment;

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 51.5 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2;

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 70.9 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection;

$$\delta_{lim} = L_{s1} / 360 = 14.444 \text{ mm}$$

Maximum deflection span 1;

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 10.835 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

CONCRETE PADSTONE

MASONRY BEARING DESIGN (BS5628)

MASONRY BEARING DESIGN TO BS5628-1:2005

TEDDS calculation version 1.0.06

Masonry details

Masonry type;

Compressive strength of unit;

Mortar designation;

Least horizontal dimension of masonry units;

Height of masonry units;

Category of masonry units;

Category of construction control ;

Partial safety factor for material strength;

Thickness of load bearing leaf;

Effective thickness of masonry wall;

Height of masonry wall;

Aggregate concrete blocks (25% or less formed voids)

$$p_{unit} = 3.6 \text{ N/mm}^2$$

iii

$$l_{unit} = 100 \text{ mm}$$

$$h_{unit} = 215 \text{ mm}$$

Category II

Normal

$$\gamma_m = 3.5$$

$$t = 100 \text{ mm}$$

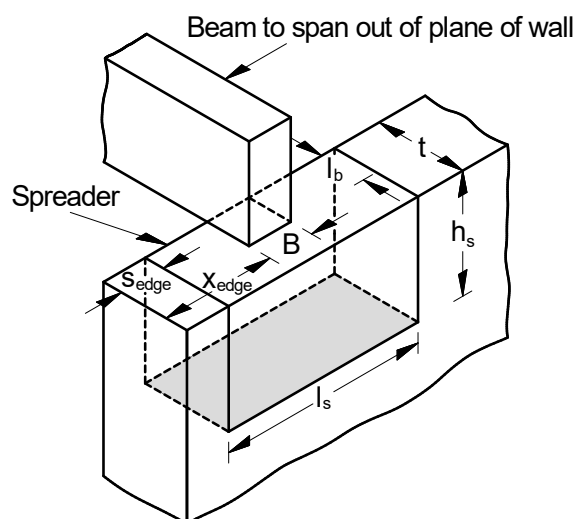
$$t_{ef} = 100 \text{ mm}$$

$$h = 2400 \text{ mm}$$

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Effective height of masonry wall;

$h_{ef} = 2400 \text{ mm}$



Bearing details

Beam spanning out of plane of wall

Width of bearing; $B = 100 \text{ mm}$
Length of bearing; $l_b = 100 \text{ mm}$
Edge distance; $X_{edge} = 100 \text{ mm}$

Compressive strength from Table 2 BS5628:Part 1 - aggregate concrete blocks (25% or less formed voids)

;Mortar designation; Mortar = "iii"
;Block compressive strength; $p_{unit} = 3.6 \text{ N/mm}^2$
;Characteristic compressive strength (Table 2c); $f_{kc} = 1.70 \text{ N/mm}^2$
;Characteristic compressive strength (Table 2d); $f_{kd} = 3.50 \text{ N/mm}^2$
;Height of solid block; $h_{unit} = 215.0 \text{ mm}$;
;Least horizontal dimension; $l_{unit} = 100.0 \text{ mm}$
Block ratio; $ratio = h_{unit} / l_{unit} = 2.2$

Ratio between 0.6 and 4.5 - OK

Characteristic compressive strength; $f_k = 3.50 \text{ N/mm}^2$

Loading details

Characteristic concentrated dead load; $G_k = 12 \text{ kN}$
Characteristic concentrated imposed load; $Q_k = 15 \text{ kN}$
Design concentrated load; $F = (G_k \times 1.4) + (Q_k \times 1.6) = 40.8 \text{ kN}$
Characteristic distributed dead load; $g_k = 0.0 \text{ kN/m}$
Characteristic distributed imposed load; $q_k = 0.0 \text{ kN/m}$
Design distributed load; $f = (g_k \times 1.4) + (q_k \times 1.6) = 0.0 \text{ kN/m}$

Masonry bearing type

Bearing type; **Type 2**
Bearing safety factor; $\gamma_{bear} = 1.50$

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Check design bearing without a spreader

Design bearing stress; $f_{ca} = F / (B \times l_b) + f / t = 4.080 \text{ N/mm}^2$

Allowable bearing stress; $f_{cp} = \gamma_{bear} \times f_k / \gamma_m = 1.500 \text{ N/mm}^2$

;NOTE - Design bearing stress exceeds allowable bearing stress, use a spreader

Spreader details

;Length of spreader; $l_s = 330 \text{ mm}$

Depth of spreader; $h_s = 215 \text{ mm}$

Edge distance; $x_{edge} = \max(0 \text{ mm}, x_{edge} - (l_s - B) / 2) = 0 \text{ mm}$

Spreader bearing type

;Bearing type; **Type 3**

Bearing safety factor; $\gamma_{bear} = 2.00$

Check design bearing with a spreader

Loading acts eccentrically within middle third – triangular stress distribution

Eccentricity of load; $e = ((l_s - B) / 2) - x_{edge} = 15 \text{ mm}$

Maximum bearing stress; $f_{ca} = F \times (1 + (6 \times e / l_s)) / (l_s \times t) + f / t = 1.574 \text{ N/mm}^2$

Allowable bearing stress; $f_{cp} = \gamma_{bear} \times f_k / \gamma_m = 2.000 \text{ N/mm}^2$

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at $0.4 \times h$ below the bearing level

Slenderness ratio; $h_{ef} / t_{ef} = 24.00$

Eccentricity at top of wall; $e_x = 0.0 \text{ mm}$

From BS5628:1 Table 7

Capacity reduction factor; $\beta = 0.61$

Length of bearing distributed at $0.4 \times h$; $l_d = 1160 \text{ mm}$

Maximum bearing stress; $f_{ca} = F / (l_d \times t) + f / t = 0.352 \text{ N/mm}^2$

Allowable bearing stress; $f_{cp} = \beta \times f_k / \gamma_m = 0.605 \text{ N/mm}^2$

PASS - Allowable bearing stress at $0.4 \times h$ below bearing level exceeds design bearing stress

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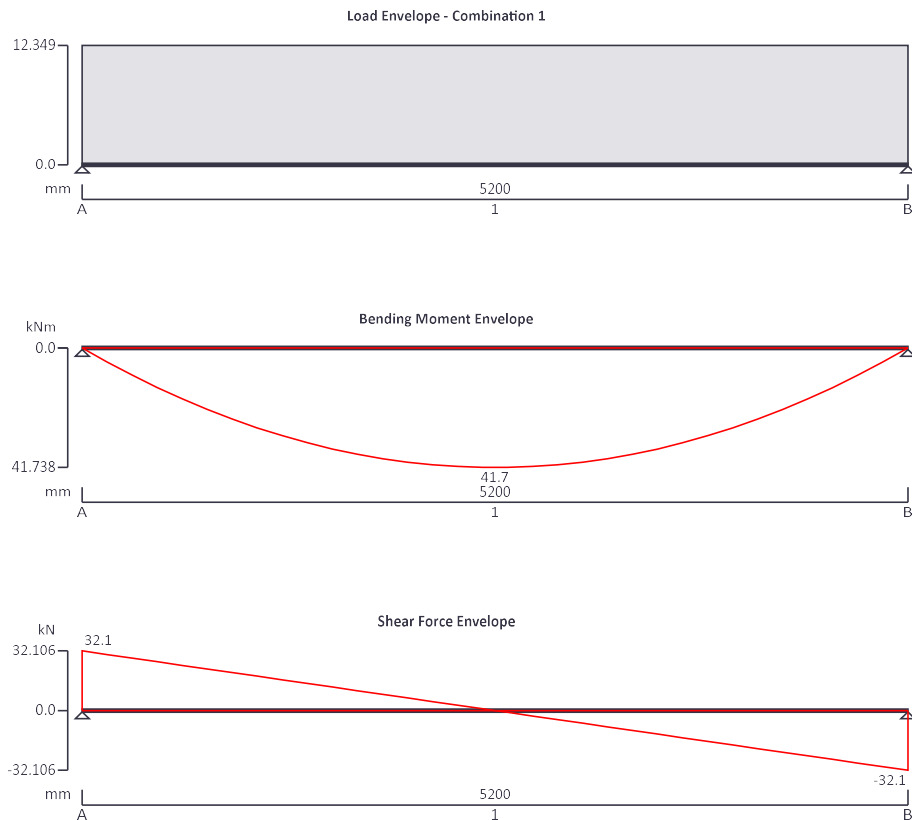
BEAM B3

STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Floor - Dead full UDL 0.56 kN/m

Floor - Imposed full UDL 1.85 kN/m

Roof - Dead full UDL 1.82 kN/m

Roof - Imposed full UDL 1.47 kN/m

Dormer wall - Dead full UDL 2.4 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

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Analysis results

Maximum moment;

Maximum shear;

Deflection;

Maximum reaction at support A;

Unfactored dead load reaction at support A;

Unfactored imposed load reaction at support A;

Maximum reaction at support B;

Unfactored dead load reaction at support B;

Unfactored imposed load reaction at support B;

Support B

$M_{max} = 41.7 \text{ kNm};$

$V_{max} = 32.1 \text{ kN};$

$\delta_{max} = 6.6 \text{ mm};$

$R_{A_{max}} = 32.1 \text{ kN};$

$R_{A_{Dead}} = 13.1 \text{ kN}$

$R_{A_{Imposed}} = 8.6 \text{ kN}$

$R_{B_{max}} = 32.1 \text{ kN};$

$R_{B_{Dead}} = 13.1 \text{ kN}$

$R_{B_{Imposed}} = 8.6 \text{ kN}$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

$M_{min} = 0 \text{ kNm}$

$V_{min} = -32.1 \text{ kN}$

$\delta_{min} = 0 \text{ mm}$

$R_{A_{min}} = 32.1 \text{ kN}$

$R_{B_{min}} = 32.1 \text{ kN}$

Section details

Section type;

Steel grade;

From table 9: Design strength p_y

Thickness of element;

Design strength;

Modulus of elasticity;

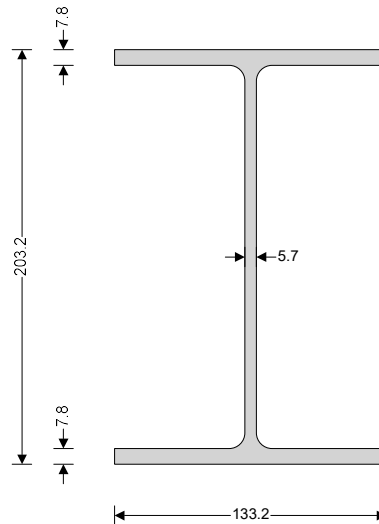
UB 203x133x25 (BS4-1)

S275

$\max(T, t) = 7.8 \text{ mm}$

$p_y = 275 \text{ N/mm}^2$

$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis;

$K_x = 1.00$

Effective length factor in minor axis;

$K_y = 1.00$

Effective length factor for lateral-torsional buckling;

$K_{LT,A} = 1.20; + 2 \times D$

$K_{LT,B} = 1.20; + 2 \times D$

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$

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Internal compression parts - Table 11

Depth of section;

$$d = 172.4 \text{ mm}$$

$$d / t = 30.2 \times \varepsilon \leq 80 \times \varepsilon;$$

Class 1 plastic

Outstand flanges - Table 11

Width of section;

$$b = B / 2 = 66.6 \text{ mm}$$

$$b / T = 8.5 \times \varepsilon \leq 9 \times \varepsilon;$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force;

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 32.1 \text{ kN}$$

$$d / t < 70 \times \varepsilon$$

Web does not need to be checked for shear buckling

Shear area;

$$A_v = t \times D = 1158 \text{ mm}^2$$

Design shear resistance;

$$P_v = 0.6 \times p_y \times A_v = 191.1 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment;

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 41.7 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2;

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 70.9 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection;

$$\delta_{lim} = L_{s1} / 360 = 14.444 \text{ mm}$$

Maximum deflection span 1;

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 6.589 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

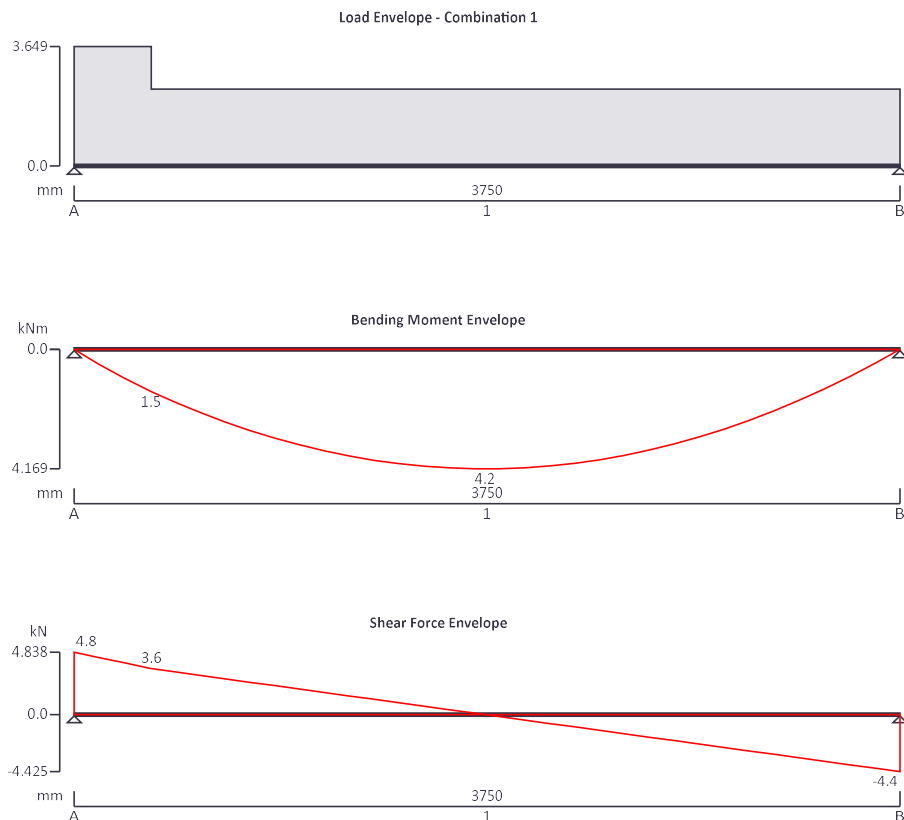
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STAIR TRIMMERS

TIMBER BEAM ANALYSIS & DESIGN (BS5268)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01



Applied loading

Beam loads

	Dead self weight of beam × 1
Floor	Dead full UDL 0.240 kN/m
Floor	Imposed full UDL 0.800 kN/m
Stud wall	Dead full UDL 1.200 kN/m
Floor	Dead partial UDL 0.300 kN/m from 0 mm to 350 mm
Floor	Imposed partial UDL 1.000 kN/m from 0 mm to 350 mm

Load combinations

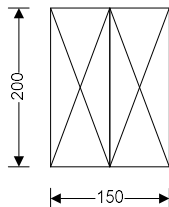
Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00

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Imposed $\times 1.00$

Analysis results

Maximum moment;	$M_{\max} = 4.169 \text{ kNm};$	$M_{\min} = 0.000 \text{ kNm}$
Design moment;	$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 4.169 \text{ kNm}$	
Maximum shear;	$F_{\max} = 4.838 \text{ kN};$	$F_{\min} = -4.425 \text{ kN}$
Design shear;	$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 4.838 \text{ kN}$	
Total load on beam;	$W_{\text{tot}} = 9.263 \text{ kN}$	
Reactions at support A;	$R_{A_{\max}} = 4.838 \text{ kN};$	$R_{A_{\min}} = 4.838 \text{ kN}$
Unfactored dead load reaction at support A;	$R_{A_{\text{Dead}}} = 3.004 \text{ kN}$	
Unfactored imposed load reaction at support A;	$R_{A_{\text{Imposed}}} = 1.834 \text{ kN}$	
Reactions at support B;	$R_{B_{\max}} = 4.425 \text{ kN};$	$R_{B_{\min}} = 4.425 \text{ kN}$
Unfactored dead load reaction at support B;	$R_{B_{\text{Dead}}} = 2.909 \text{ kN}$	
Unfactored imposed load reaction at support B;	$R_{B_{\text{Imposed}}} = 1.516 \text{ kN}$	



Timber section details

Breadth of sections;	$b = 75 \text{ mm}$
Depth of sections;	$h = 200 \text{ mm}$
Number of sections in member;	$N = 2$
Overall breadth of member;	$b_b = N \times b = 150 \text{ mm}$
Timber strength class;	C16

Member details

Service class of timber;	1
Load duration;	Long term
Length of span;	$L_{s1} = 3750 \text{ mm}$
Length of bearing;	$L_b = 100 \text{ mm}$

Section properties

Cross sectional area of member;	$A = N \times b \times h = 30000 \text{ mm}^2$
Section modulus;	$Z_x = N \times b \times h^2 / 6 = 1000000 \text{ mm}^3$
	$Z_y = h \times (N \times b)^2 / 6 = 750000 \text{ mm}^3$
Second moment of area;	$I_x = N \times b \times h^3 / 12 = 100000000 \text{ mm}^4$
	$I_y = h \times (N \times b)^3 / 12 = 56250000 \text{ mm}^4$
Radius of gyration;	$i_x = \sqrt{(I_x / A)} = 57.7 \text{ mm}$
	$i_y = \sqrt{(I_y / A)} = 43.3 \text{ mm}$

Modification factors

Duration of loading - Table 17;	$K_3 = 1.00$
Bearing stress - Table 18;	$K_4 = 1.00$
Total depth of member - cl.2.10.6;	$K_7 = (300 \text{ mm} / h)^{0.11} = 1.05$
Load sharing - cl.2.10.11;	$K_8 = 1.10$
Minimum modulus of elasticity - Table 20;	$K_9 = 1.14$

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Lateral support - cl.2.10.8

No lateral support

Permissible depth-to-breadth ratio - Table 19;

2.00

Actual depth-to-breadth ratio;

$h / (N \times b) = \mathbf{1.33}$

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane);

$\sigma_{c_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = \mathbf{2.420 \text{ N/mm}^2}$

Applied bearing stress;

$\sigma_{c_a} = R_{A_max} / (N \times b \times L_b) = \mathbf{0.323 \text{ N/mm}^2}$

$\sigma_{c_a} / \sigma_{c_adm} = \mathbf{0.133}$

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress;

$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{6.096 \text{ N/mm}^2}$

Applied bending stress;

$\sigma_{m_a} = M / Z_x = \mathbf{4.169 \text{ N/mm}^2}$

$\sigma_{m_a} / \sigma_{m_adm} = \mathbf{0.684}$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress;

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

Applied shear stress;

$\tau_a = 3 \times F / (2 \times A) = \mathbf{0.242 \text{ N/mm}^2}$

$\tau_a / \tau_{adm} = \mathbf{0.328}$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection;

$E = E_{min} \times K_9 = \mathbf{6612 \text{ N/mm}^2}$

Permissible deflection;

$\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) = \mathbf{11.250 \text{ mm}}$

Bending deflection;

$\delta_{b_s1} = \mathbf{9.252 \text{ mm}}$

Shear deflection;

$\delta_{v_s1} = \mathbf{0.404 \text{ mm}}$

Total deflection;

$\delta_a = \delta_{b_s1} + \delta_{v_s1} = \mathbf{9.656 \text{ mm}}$

$\delta_a / \delta_{adm} = \mathbf{0.858}$

PASS - Total deflection is less than permissible deflection

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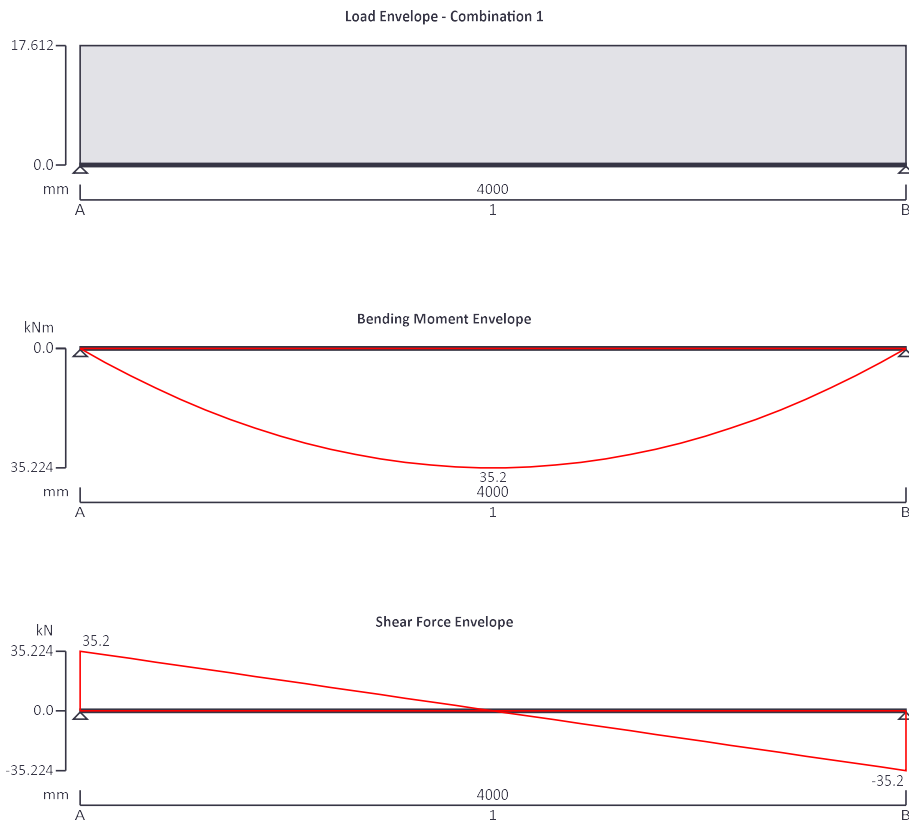
BEAM B4 - SUPPORTING INNER LEAF

STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Roof extg - Dead full UDL 0.28 kN/m

Roof extg - Imposed full UDL 0.225 kN/m

Wall - Dead full UDL 5.4 kN/m

Floor - Dead full UDL 1.32 kN/m

Floor - Imposed full UDL 4.4 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

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Analysis results

Maximum moment;

Maximum shear;

Deflection;

Maximum reaction at support A;

Unfactored dead load reaction at support A;

Unfactored imposed load reaction at support A;

Maximum reaction at support B;

Unfactored dead load reaction at support B;

Unfactored imposed load reaction at support B;

Support B

$M_{max} = 35.2 \text{ kNm};$

$V_{max} = 35.2 \text{ kN};$

$\delta_{max} = 6.7 \text{ mm};$

$R_{A_max} = 35.2 \text{ kN};$

$R_{A_Dead} = 14.6 \text{ kN}$

$R_{A_Imposed} = 9.3 \text{ kN}$

$R_{B_max} = 35.2 \text{ kN};$

$R_{B_Dead} = 14.6 \text{ kN}$

$R_{B_Imposed} = 9.3 \text{ kN}$

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

$M_{min} = 0 \text{ kNm}$

$V_{min} = -35.2 \text{ kN}$

$\delta_{min} = 0 \text{ mm}$

$R_{A_min} = 35.2 \text{ kN}$

$R_{B_min} = 35.2 \text{ kN}$

Section details

Section type;

Steel grade;

From table 9: Design strength p_y

Thickness of element;

Design strength;

Modulus of elasticity;

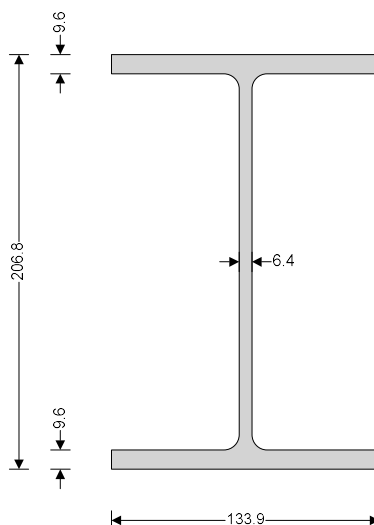
UB 203x133x30 (BS4-1)

S275

$\max(T, t) = 9.6 \text{ mm}$

$p_y = 275 \text{ N/mm}^2$

$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis;

$K_x = 1.00$

Effective length factor in minor axis;

$K_y = 1.00$

Effective length factor for lateral-torsional buckling;

$K_{LT,A} = 1.20; + 2 \times D$

$K_{LT,B} = 1.20; + 2 \times D$

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$

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Internal compression parts - Table 11

Depth of section; $d = 172.4$ mm
 $d / t = 26.9 \times \varepsilon \leq 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = 67$ mm
 $b / T = 7.0 \times \varepsilon \leq 9 \times \varepsilon$; Class 1 plastic
Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 35.2$ kN
 $d / t < 70 \times \varepsilon$
Web does not need to be checked for shear buckling
 Shear area; $A_v = t \times D = 1324$ mm²
 Design shear resistance; $P_v = 0.6 \times p_y \times A_v = 218.4$ kN
PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 35.2$ kNm
 Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 86.5$ kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.2 \times L_{s1} + 2 \times D = 5214$ mm
 Slenderness ratio; $\lambda = L_E / r_{yy} = 164.322$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = 0.881$
 Torsional index; $x = 21.493$
 Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.711$
 Ratio - cl.4.3.6.9; $\beta_W = 1.000$
 Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_W} = 102.923$
 Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$
 $\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} = 7.0$
 Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.480$
 Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = 191$ N/mm²
 $\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 278.9$ N/mm²
 Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 120$ N/mm²

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB; $m_{LT} = 1.000$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = 37.7$ kNm
 $M_b / m_{LT} = 37.7$ kNm
PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 = 11.111$ mm
 Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 6.693$ mm

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PASS - Maximum deflection does not exceed deflection limit

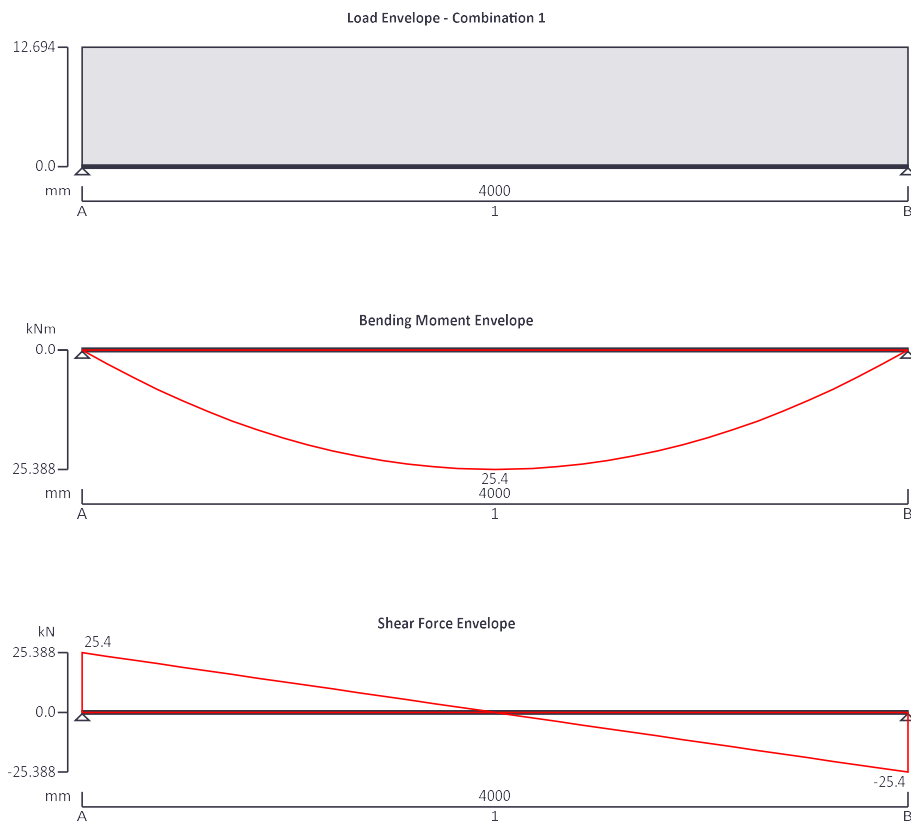
BEAM B5 - SUPPORTING OUTER LEAF

STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Lean to roof - Dead full UDL 1.721 kN/m

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Load combinations

Load combination 1

Support A

Dead $\times$ 1.40

Imposed $\times$ 1.60

Dead $\times$ 1.40

Imposed $\times$ 1.60

Support B

Dead $\times$ 1.40

Imposed $\times$ 1.60

Analysis results

Maximum moment;

$M_{\max} = 25.4$ kNm;

$M_{\min} = 0$ kNm

Maximum shear;

$V_{\max} = 25.4$ kN;

$V_{\min} = -25.4$ kN

Deflection;

$\delta_{\max} = 6.2$ mm;

$\delta_{\min} = 0$ mm

Maximum reaction at support A;

$R_{A_{\max}} = 25.4$ kN;

$R_{A_{\min}} = 25.4$ kN

Unfactored dead load reaction at support A;

$R_{A_{\text{Dead}}} = 14.9$ kN

Unfactored imposed load reaction at support A;

$R_{A_{\text{Imposed}}} = 2.8$ kN

Maximum reaction at support B;

$R_{B_{\max}} = 25.4$ kN;

$R_{B_{\min}} = 25.4$ kN

Unfactored dead load reaction at support B;

$R_{B_{\text{Dead}}} = 14.9$ kN

Unfactored imposed load reaction at support B;

$R_{B_{\text{Imposed}}} = 2.8$ kN

Section details

Section type;

UB 203x133x25 (BS4-1)

Steel grade;

S275

From table 9: Design strength p_y

Thickness of element;

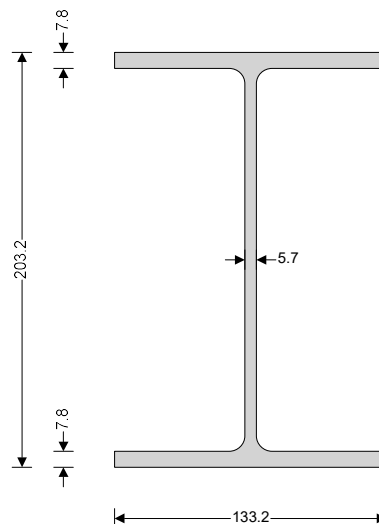
$\max(T, t) = 7.8$ mm

Design strength;

$p_y = 275$ N/mm²

Modulus of elasticity;

$E = 205000$ N/mm²



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis;

$K_x = 1.00$

Effective length factor in minor axis;

$K_y = 1.00$

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Effective length factor for lateral-torsional buckling; $K_{LT,A} = 1.20; + 2 \times D$

$$K_{LT,B} = 1.20; + 2 \times D$$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$$

Internal compression parts - Table 11

Depth of section;

$$d = 172.4 \text{ mm}$$

$$d / t = 30.2 \times \varepsilon \leq 80 \times \varepsilon; \quad \text{Class 1 plastic}$$

Outstand flanges - Table 11

Width of section;

$$b = B / 2 = 66.6 \text{ mm}$$

$$b / T = 8.5 \times \varepsilon \leq 9 \times \varepsilon; \quad \text{Class 1 plastic}$$

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force;

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 25.4 \text{ kN}$$

$$d / t < 70 \times \varepsilon$$

Web does not need to be checked for shear buckling

Shear area;

$$A_v = t \times D = 1158 \text{ mm}^2$$

Design shear resistance;

$$P_v = 0.6 \times p_y \times A_v = 191.1 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment;

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 25.4 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2;

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 70.9 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling;

$$L_E = 1.2 \times L_{s1} + 2 \times D = 5206 \text{ mm}$$

Slenderness ratio;

$$\lambda = L_E / r_{yy} = 167.836$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter;

$$u = 0.877$$

Torsional index;

$$x = 25.605$$

Slenderness factor;

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.751$$

Ratio - cl.4.3.6.9;

$$\beta_W = 1.000$$

Equivalent slenderness - cl.4.3.6.7;

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_W} = 110.494$$

Limiting slenderness - Annex B.2.2;

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$$

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant;

$$\alpha_{LT} = 7.0$$

Perry factor;

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.533$$

Euler stress;

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 165.7 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 264.5 \text{ N/mm}^2$$

Bending strength - Annex B.2.1;

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 108.3 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB;

$$m_{LT} = 1.000$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment;

$$M_b = p_b \times S_{xx} = 27.9 \text{ kNm}$$

$$M_b / m_{LT} = 27.9 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

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Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection;

$$\delta_{lim} = L_{s1} / 360 = 11.111 \text{ mm}$$

Maximum deflection span 1;

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 6.161 \text{ mm}$$

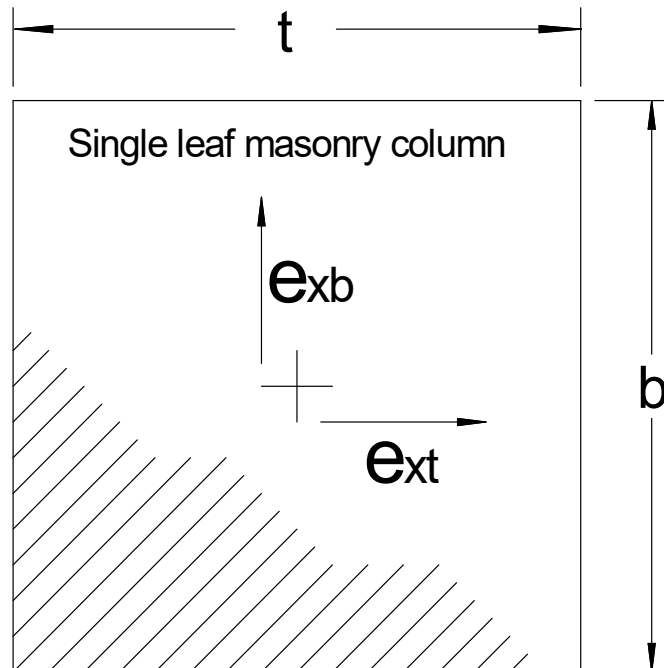
PASS - Maximum deflection does not exceed deflection limit

MASONRY PIER SUPPORTING NEW BEAMS

MASONRY COLUMN DESIGN (BS5628)

VERTICAL LOADING RECTANGULAR COLUMN (BS5628-1:2005)

TEDDS calculation version 1.0.02



Compressive strength from Table 2 BS5628:Part 1 - aggregate concrete blocks (25% or less formed voids)

;Mortar designation;	Mortar = "iii"
;Block compressive strength;	$p_{unit} = 7.3 \text{ N/mm}^2$
;;;;;Characteristic compressive strength (Table 2c);	$f_{kc} = 3.20 \text{ N/mm}^2$
;;;;;Characteristic compressive strength (Table 2d);	$f_{kd} = 6.40 \text{ N/mm}^2$
;Height of solid block;	$h_{unit} = 215.0 \text{ mm}$;
;Least horizontal dimension;	$l_{unit} = 100.0 \text{ mm}$
Block ratio;	ratio = $h_{unit} / l_{unit} = 2.2$

Ratio between 0.6 and 4.5 - OK

Characteristic compressive strength;	$f_k = 6.40 \text{ N/mm}^2$
;Column width;	$b = 200 \text{ mm}$
;Column thickness;	$t = 330 \text{ mm}$
;Column height;	$h = 2.40 \text{ m}$
Column slenderness - minor axis (Clause 24.1);	$\lambda_t = h/t = 7.27$

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Column slenderness - major axis (Clause 24.1);

$$\lambda_{lb} = h/b = \mathbf{12.00}$$

Maximum slenderness;

$$\lambda_{\max} = \max(\lambda_t, \lambda_b) = \mathbf{12.00}$$

Slenderness < 27 - OK

;Partial safety factor for material (Table 4);

$$\gamma_m = \mathbf{3.5}$$

Load eccentricity

;Eccentricity of applied load about minor axis;

$$e_{xt} = \mathbf{25.0 \text{ mm}}$$

;Eccentricity of applied load about major axis;

$$e_{xb} = \mathbf{25.0 \text{ mm}}$$

Capacity reduction factor: minor axis

Eccentricity due to slenderness;

$$e_{at} = \max(0 \text{ mm}, t \times (((h/t)^2/2400)-0.015)) = \mathbf{2.3 \text{ mm}}$$

Design eccentricity;

$$e_{tt} = 0.6 \times \max(\text{abs}(e_{xt}), 0.05 \times t) + e_{at} = \mathbf{17.3 \text{ mm}}$$

$$e_{mt} = \max(\text{abs}(e_{xt}), e_{tt}) = \mathbf{25.0 \text{ mm}};$$

Capacity reduction factors;

$$\beta_{tcalc} = \max(0, 1.1 \times (1 - (2 \times e_{mt} / t))) = \mathbf{0.93}$$

$$\beta_{\max} = 1.0$$

$$\beta_t = \min(\beta_{tcalc}, \beta_{\max}) = \mathbf{0.93}$$

Capacity reduction factor: major axis

Eccentricity due to slenderness;

$$e_{ab} = \max(0 \text{ mm}, b \times (((h/b)^2/2400)-0.015)) = \mathbf{9.0 \text{ mm}}$$

Design eccentricity;

$$e_{tb} = 0.6 \times \max(\text{abs}(e_{xb}), 0.05 \times b) + e_{ab} = \mathbf{24.0 \text{ mm}}$$

$$e_{mb} = \max(\text{abs}(e_{xb}), e_{tb}) = \mathbf{25.0 \text{ mm}};$$

Capacity reduction factors;

$$\beta_{bcalc} = \max(0, 1.1 \times (1 - (2 \times e_{mb} / b))) = \mathbf{0.83}$$

$$\beta_{\max} = 1.0$$

$$\beta_b = \min(\beta_{bcalc}, \beta_{\max}) = \mathbf{0.83}$$

Minimum capacity reduction factor;

$$\beta_{\min} = \min(\beta_t, \beta_b) = \mathbf{0.83}$$

Design vertical load resistance

Compressive strength correction factor

Plan area of column;

$$A = t \times b = \mathbf{0.07 \text{ m}^2}$$

For small plan area (Clause 19.1.2);

$$c = \min(1.0, 0.7 + (1.5 \text{ m}^2) \times A) = \mathbf{0.80}$$

Design vertical load resistance;

$$DCLR = \beta_{\min} \times t \times b \times c \times f_k / \gamma_m = \mathbf{79.553 \text{ kN}}$$

;Applied factored vertical load on column;

$$V = \mathbf{67.000 \text{ kN}}$$

Column - OK

;

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CHECK MASONRY PIER FOR LATERAL STABILITY

LATERAL LOADING

Consider a wind loading of 0.60 kN/m².

Height of pier approx 2.40m

Average area of wind loading into pier = 13.00m²

Force into top pier = 0.60kN/m² x 13.00m² = 7.80 kN

Factored force = 1.40 x 7.80 = 10.92 kN

Force into each pier = 10.92 / 2 = 5.46 kN (Shear Applied)

Moment at each pier = 5.46 x 2.40 = 13.10 kN-m

From,

$P/A \pm M/Z$

Where:

P = 70kN (Factored)

A = 200 x 330 = 66000 mm²

M = 13.10 kN-m

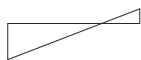
$Z = bd^2 / 6 = \{200 \times (330)^2\} / 6 = 3630000 \text{ mm}^3$

Hence,

$$(70 \times 10^3 / 66000) + (13.10 \times 10^6 / 3.63 \times 10^6) = 1.06 + 3.61 = +4.67 \text{ N/mm}^2$$

$$= 1.06 - 3.61 = -2.55 \text{ N/mm}^2$$

Stress Diagram



Thus, Design depth of pier to be 221mm (for consideration of its shear capacity), as the remainder of the pier 109mm is effectively in tension.

Lateral Stability Check for Masonry: From BS 5628-1 Section 3 clause 25

Characteristic shear strength $f_v = (0.35 + 0.60g_a) < 1.75 \text{ N/mm}^2$ for walls built in mortar designation (i) and (ii)

OR $f_v = (0.15 + 0.60g_a) < 1.40 \text{ N/mm}^2$ for walls built in mortar designation (iii) and (iv).

Using

$$f_v = (0.15 + 0.60g_a) = 0.15 + (0.60 \times (2.84 / 2)) = 1.01 \text{ N/mm}^2 < 1.40 \text{ N/mm}^2$$

Thus Shear capacity of pier $1.01 \times 221 \times 200 / 1000 = 44.64 \text{ kN} > 5.46 \text{ kN}$ Pass

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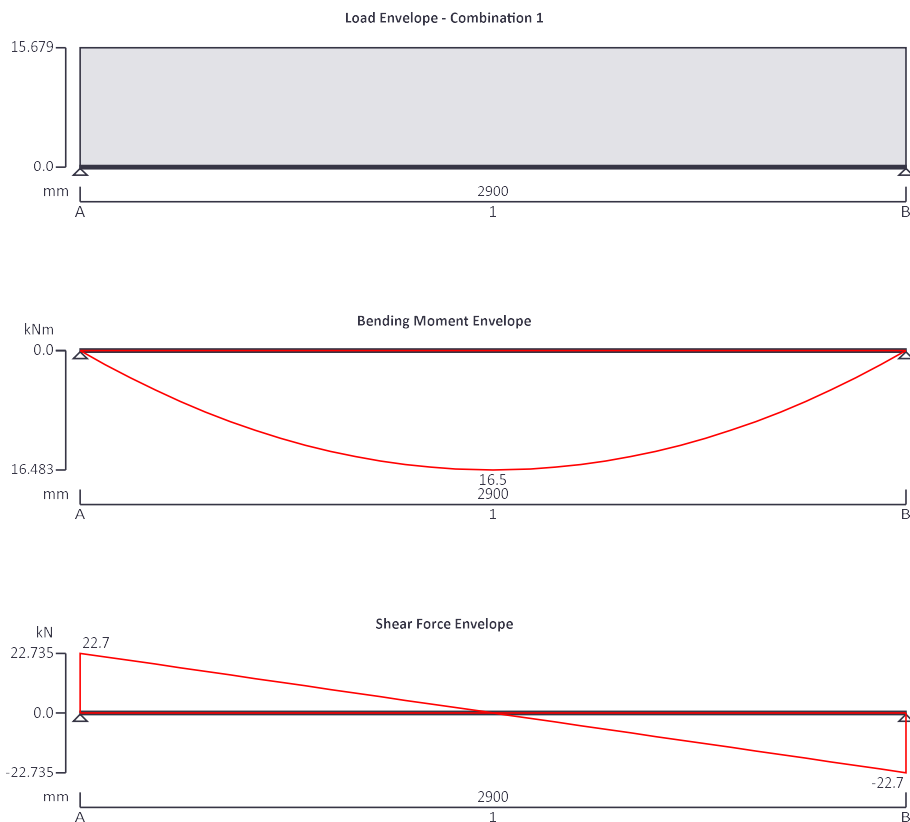
BEAM B6

STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Floor - Dead full UDL 2.3 kN/m

Floor - Imposed full UDL 7.65 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

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Analysis results

Maximum moment;

Maximum shear;

Deflection;

Maximum reaction at support A;

Unfactored dead load reaction at support A;

Unfactored imposed load reaction at support A;

Maximum reaction at support B;

Unfactored dead load reaction at support B;

Unfactored imposed load reaction at support B;

Support B

$M_{max} = 16.5 \text{ kNm};$

$V_{max} = 22.7 \text{ kN};$

$\delta_{max} = 4.1 \text{ mm};$

$R_{A_max} = 22.7 \text{ kN};$

$R_{A_Dead} = 3.6 \text{ kN}$

$R_{A_Imposed} = 11.1 \text{ kN}$

$R_{B_max} = 22.7 \text{ kN};$

$R_{B_Dead} = 3.6 \text{ kN}$

$R_{B_Imposed} = 11.1 \text{ kN}$

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

$M_{min} = 0 \text{ kNm}$

$V_{min} = -22.7 \text{ kN}$

$\delta_{min} = 0 \text{ mm}$

$R_{A_min} = 22.7 \text{ kN}$

$R_{B_min} = 22.7 \text{ kN}$

Section details

Section type;

Steel grade;

From table 9: Design strength p_y

Thickness of element;

Design strength;

Modulus of elasticity;

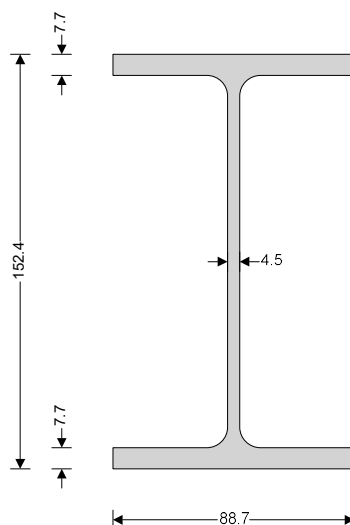
UB 152x89x16 (BS4-1)

S275

$\max(T, t) = 7.7 \text{ mm}$

$p_y = 275 \text{ N/mm}^2$

$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis;

$K_x = 1.00$

Effective length factor in minor axis;

$K_y = 1.00$

Effective length factor for lateral-torsional buckling;

$K_{LT,A} = 1.00; + 2 \times D$

$K_{LT,B} = 1.00; + 2 \times D$

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$

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Internal compression parts - Table 11

Depth of section; $d = 121.8 \text{ mm}$
 $d / t = 27.1 \times \varepsilon \leq 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = 44.4 \text{ mm}$
 $b / T = 5.8 \times \varepsilon \leq 9 \times \varepsilon$; Class 1 plastic
Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 22.7 \text{ kN}$
 $d / t < 70 \times \varepsilon$
Web does not need to be checked for shear buckling
 Shear area; $A_v = t \times D = 686 \text{ mm}^2$
 Design shear resistance; $P_v = 0.6 \times p_y \times A_v = 113.2 \text{ kN}$
PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 16.5 \text{ kNm}$
 Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 33.9 \text{ kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1} + 2 \times D = 3205 \text{ mm}$
 Slenderness ratio; $\lambda = L_E / r_{yy} = 152.493$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = 0.890$
 Torsional index; $x = 19.566$
 Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.705$
 Ratio - cl.4.3.6.9; $\beta_w = 1.000$
 Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 95.739$
 Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$
 $\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} = 7.0$
 Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.430$
 Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = 220.7 \text{ N/mm}^2$
 $\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 295.3 \text{ N/mm}^2$
 Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 132.5 \text{ N/mm}^2$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = 12.4 \text{ kNm}$
 Moment at centre-line of segment; $M_3 = 16.5 \text{ kNm}$
 Moment at three quarter point of segment; $M_4 = 12.4 \text{ kNm}$
 Maximum moment in segment; $M_{abs} = 16.5 \text{ kNm}$
 Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = 16.5 \text{ kNm}$
 Equivalent uniform moment factor for lateral-torsional buckling;
 $m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.925$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = 16.3 \text{ kNm}$

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$$M_b / M_{LT} = 17.7 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection;

$$\delta_{lim} = L_{s1} / 360 = 8.056 \text{ mm}$$

Maximum deflection span 1;

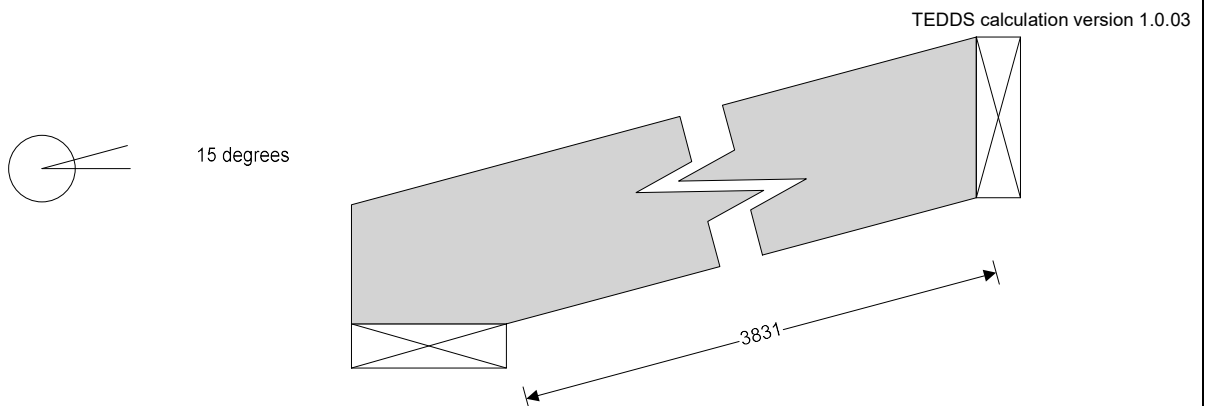
$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 4.119 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

LEAN TO ROOF RAFTERS

TIMBER RAFTER DESIGN (BS5268)

TIMBER RAFTER DESIGN (BS5268-2:2002)



Rafter details

Breadth of timber sections;

$$b = 50 \text{ mm}$$

Depth of timber sections;

$$h = 175 \text{ mm}$$

Rafter spacing;

$$s = 400 \text{ mm}$$

Rafter slope;

$$\alpha = 15.0 \text{ deg}$$

Clear span of rafter on horizontal;

$$L_{clh} = 3700 \text{ mm}$$

Clear span of rafter on slope;

$$L_{cl} = L_{clh} / \cos(\alpha) = 3831 \text{ mm}$$

Rafter span;

Single span

Timber strength class;

C16

Section properties

Cross sectional area of rafter;

$$A = b \times h = 8750 \text{ mm}^2$$

Section modulus;

$$Z = b \times h^2 / 6 = 255208 \text{ mm}^3$$

Second moment of area;

$$I = b \times h^3 / 12 = 22330729 \text{ mm}^4$$

Radius of gyration;

$$r = \sqrt{I / A} = 50.5 \text{ mm}$$

Loading details

Rafter self weight;

$$F_j = b \times h \times \rho_{char} \times g_{acc} = 0.03 \text{ kN/m}$$

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Dead load on slope;
Imposed load on plan;
Imposed point load;

$$F_d = 0.93 \text{ kN/m}^2$$

$$F_u = 0.75 \text{ kN/m}^2$$

$$F_p = 0.90 \text{ kN}$$

Modification factors

Section depth factor;
Load sharing factor;

$$K_7 = (300 \text{ mm} / h)^{0.11} = 1.06$$

$$K_8 = 1.10$$

Consider long term load condition

Load duration factor;
Total UDL perpendicular to rafter;
Notional bearing length;
Effective span;

$$K_3 = 1.00$$

$$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = 0.385 \text{ kN/m}$$

$$L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 6 \text{ mm}$$

$$L_{eff} = L_{cl} + L_b = 3837 \text{ mm}$$

Check bending stress

Bending stress parallel to grain;
Permissible bending stress;
Applied bending stress;

$$\sigma_m = 5.300 \text{ N/mm}^2$$

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 6.186 \text{ N/mm}^2$$

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 2.776 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain;
Minimum modulus of elasticity;
Compression member factor;
Permissible compressive stress;
Applied compressive stress;

$$\sigma_c = 6.800 \text{ N/mm}^2$$

$$E_{min} = 5800 \text{ N/mm}^2$$

$$K_{12} = 0.54$$

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 4.037 \text{ N/mm}^2$$

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.383 \text{ N/mm}^2$$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress;
Euler coefficient;
Combined axial compression and bending check;

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 9.925 \text{ N/mm}^2$$

$$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.969$$

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.558; < 1$$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain;
Permissible shear stress;
Applied shear stress;

$$\tau = 0.670 \text{ N/mm}^2$$

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

$$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = 0.127 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection;
Bending deflection;
Shear deflection;
Total deflection;

$$\delta_{adm} = 0.003 \times L_{eff} = 11.510 \text{ mm}$$

$$\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = 5.528 \text{ mm}$$

$$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = 0.177 \text{ mm}$$

$$\delta_{max} = \delta_b + \delta_s = 5.704 \text{ mm}$$

PASS - Total deflection within permissible limits

Consider medium term load condition

Load duration factor;
Total UDL perpendicular to rafter;
Notional bearing length;
Effective span;

$$K_3 = 1.25$$

$$F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha) = 0.665 \text{ kN/m}$$

$$L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 11 \text{ mm}$$

$$L_{eff} = L_{cl} + L_b = 3841 \text{ mm}$$

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Check bending stress

Bending stress parallel to grain;

$$\sigma_m = 5.300 \text{ N/mm}^2$$

Permissible bending stress;

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 7.733 \text{ N/mm}^2$$

Applied bending stress;

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 4.805 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain;

$$\sigma_c = 6.800 \text{ N/mm}^2$$

Minimum modulus of elasticity;

$$E_{min} = 5800 \text{ N/mm}^2$$

Compression member factor;

$$K_{12} = 0.49$$

Permissible compressive stress;

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 4.594 \text{ N/mm}^2$$

Applied compressive stress;

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.662 \text{ N/mm}^2$$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress;

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 9.902 \text{ N/mm}^2$$

Euler coefficient;

$$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.951$$

Combined axial compression and bending check;

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.798; < 1$$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain;

$$\tau = 0.670 \text{ N/mm}^2$$

Permissible shear stress;

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

Applied shear stress;

$$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = 0.219 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection;

$$\delta_{adm} = 0.003 \times L_{eff} = 11.523 \text{ mm}$$

Bending deflection;

$$\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = 9.591 \text{ mm}$$

Shear deflection;

$$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = 0.306 \text{ mm}$$

Total deflection;

$$\delta_{max} = \delta_b + \delta_s = 9.896 \text{ mm}$$

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor;

$$K_3 = 1.50$$

Total UDL perpendicular to rafter;

$$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = 0.385 \text{ kN/m}$$

Notional bearing length;

$$L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 10 \text{ mm}$$

Effective span;

$$L_{eff} = L_{cl} + L_b = 3840 \text{ mm}$$

Check bending stress

Bending stress parallel to grain;

$$\sigma_m = 5.300 \text{ N/mm}^2$$

Permissible bending stress;

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 9.279 \text{ N/mm}^2$$

Applied bending stress;

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) + F_p \times \cos(\alpha) \times L_{eff} / (4 \times Z) = 6.051 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain;

$$\sigma_c = 6.800 \text{ N/mm}^2$$

Minimum modulus of elasticity;

$$E_{min} = 5800 \text{ N/mm}^2$$

Compression member factor;

$$K_{12} = 0.45$$

Permissible compressive stress;

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 5.025 \text{ N/mm}^2$$

Applied compressive stress;

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) + F_p \times \sin(\alpha) / A = 0.410 \text{ N/mm}^2$$

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PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 9.906 \text{ N/mm}^2$

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.972$

Combined axial compression and bending check; $\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.752; < 1$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; $\tau = 0.670 \text{ N/mm}^2$

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8 = 1.106 \text{ N/mm}^2$

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A) = 0.276 \text{ N/mm}^2$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff} = 11.521 \text{ mm}$

Bending deflection; $\delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha) / 48) / (E_{mean} \times I) = 10.768 \text{ mm}$

Shear deflection; $\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha)) / (5 \times E_{mean} \times A) = 0.385 \text{ mm}$

Total deflection; $\delta_{max} = \delta_b + \delta_s = 11.153 \text{ mm}$

PASS - Total deflection within permissible limits