Structural Calculations

Project: Double storey rear & side extension

Location: 22 Sequoia Park, Pinner, HA5 4BS

Structural Engineer SYED RIZVI

GENERAL CONSTRUCTION NOTES

All existing load bearing walls, foundations and structural elements are to be exposed and checked for adequacy at the commencement of the works by the building insp-ctor. All trial holes are to be excavated and inspected by the building inspector prior to commencement.

Any deviation from the assumptions made in the calculations are to be reported to the Architect/Engineer as soon as practical after being found.
The exsiting building is assumed to be in good structural condition and any defects are to be reported to the Architect/Engineer as soon as possible. The temporary stability the structure is the responsibility of the builder during all stages of the construction.
All new timber is to be C16 grade unless noted otherwise in the calculations and all structural timber is to be suitably treated against decay and insect attack.
These calculations are to be read in conjuction with all relevant Architects and Engi-neers details and specifications. Full building regulations approval should be obtained prior to commencement of works on site and any works carried out prior to approval is done so at the contractor's risk.

The works are to be carried out the approval and satsfaction of the building inspect-or and to accepted good building practice and in full compliance with all relevant British Standards and Codes of Practice. These calculations are for building regulati-on purpose and do not constitute full working drawings/details for the project.
All beam lengths used in these calculations are design lengths and are not to be used for ordering. All beams/joist lengths are to be taken to suit site dimensions.
Foundation concret to be minimum C20 mix in accordance with Building Regulations Approved Document A. Depth of all new foundations to be determined on site to the approval of BCO.

THESE CALCULATIONS HAVE BEEN PREPARED BASED ON ARCHITECTS PLANS AND SEC-TIONS. ANY CONNECTIONS OR DETAILS SHOWN ARE FOR GUIDANCE ONLY.
FINAL CONNECTION DETAILS TO BE DESIGNED STEEL BY FABRICATOR BASED ON THE REACTIONS GIVEN IN THE CALCULATIONS BUT TO SUIT THE SITE DIMENSIONS AND RELATIVE LEVELS THAT WILL BE REQUIRED DURING THE CONSTRUCTION STAGE.
UNLESS STATED OTHERWISE IN THE CALCULATIONS ALL BOLTED SPLICES TO BEAMS ARE POSITIONED AT THIRD LENGTH POINTS. MID SPAN SPLICES WIL NOT BE ACCEPTED.

Job			
Author	Date	Revision	Sheet No.
			1

Ref	Calculations	Output
BS 6399	**Derivation of member loadings**	

Flat Roof

Stone chips and bitumen	0.20	kN/m²
Three layers of felt	0.10	kN/m²
Decking	0.10	kN/m²
Joists	0.15	kN/m²
Finishes & Insulation	0.20	kN/m²
Imposed load due to snow	0.75	kN/m²
	1.56	kN/m²

Pitched Roof

Tiles	0.70	kN/m²
Rafters+battens+insulation	0.20	kN/m²
	0.90	
above on plan = 0.9/Cos35 =	1.10	kN/m²
Plaster board + skim	0.20	kN/m²
Imposed load on pitched roof	0.70	kN/m²
	1.83	kN/m²

Stud Partition

Framing+insulation+sheathing+	0.25	
lining Skim	0.25	
	0.50	kN/m²

Timber Floor

Joists+Boarding+Ceiling	0.50	kN/m²
Imposed Load	1.50	kN/m²
	2.00	kN/m²

Cavity Wall

102mm Bricks	2.25	kN/m²
100mm Blocks	1.15	kN/m²
	3.40	kN/m²

Ceiling Load	1.00	kN/m²
Dormer Load	1.00	kN/m²

Combined Load Factor = (1.4+1.6)/2 =1.5

Note: Calculations to be approved by the local authority before work commences

22 Sequoia Park, Pinner, HA5 4BS

① Beam 1

Length = 5100 mm

Dead Loads

Floor $0.54\ kN/m^2 \times \left[\left(3.4/2\right) + \left(5/2\right)\right] = 2.268\ kN/m$

Brick wall $2.25\ kN/m^2 \times 2.5 = 5.625\ kN/m$

Pitched roof, $1.08\ kN/m^2 \times \left[3.3/2 + 3.3/2\right] = 3.78\ kN/m$

Total = 11.673 kN/m

Live Loads

Floor $1.5\ kN/m^2 \times 4.2 = 6.3\ kN/m$

Pitched roof, $0.75\ kN/m^2 \times 3.5 = 2.625\ kN/m$

Total = 8.925 kN/m

∴ use 203x203x52 UC

② Beam 2

Length = 8m

Dead Loads

Floor $0.54\ kN/m^2 \times (5/2 + 1/2) = 1.62\ kN/m$

Cavity wall $3.4\ kN/m^2 \times 3 = 10.2\ kN/m$

Pitched roof, $1.08\ kN/m^2 \times 1/2 = 0.54\ kN/m$

Flat roof, $0.81\ kN/m^2 \times 2/2 = 0.81\ kN/m$

Total = 13.7 kN/m

Live loads

Floor $1.5\ kN/m^2 \times 3 = 4.5\ kN/m$

Pitch roof $0.75\ kN/m^2 \times 1/2 = 0.375\ kN/m$

Flat roof $0.75\ kN/m^2\ 2/2 = 0.75\ kN/m$

Total = 5.625 kN/m

∴ USE 254x254x107 UC

Beam 3

Length = 3500mm

Dead loads

Floor $0.54\ kN/m^2 \times (1/2) = 0.27\ kN/m$

Cavity wall $3.4\ kN/m^2 \times 3 = 10.2\ kN/m$

Pitched roof $1.08\ kN/m^2 \times (1/2 + 3.6/2) = 2.3\ kN/m$

Total = $12.77\ kN/m$

Live loads

Floor $1.5\ kN/m^2 \times (1/2) = 0.75\ kN/m$

Pitched roof $0.75\ kN/m^2 \times (1/2 + 3.6/2) = 1.725\ kN/m$

Total = $2.475\ kN/m$

∴ use 152 x 152 x 30 UC

Beam 4

Length = 8m

Dead loads

Flat roof $0.81\ kN/m^2 \times (2/2) = 0.81\ kN/m$

Pitched roof $1.08\ kN/m^2 \times (3.3/2) = 1.782\ kN/m$

Total = $2.592\ kN/m$

Live loads

Flat roof $0.75\ kN/m^2 \times (2/2) = 0.75\ kN/m$

Pitched roof $0.75\ kN/m^2 \times (3.3/2) = 1.238\ kN/m$

Total = $1.988\ kN/m$

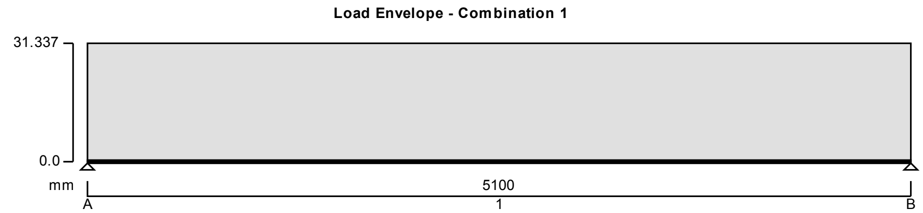
∴ use 203 x 203 x 46 UC

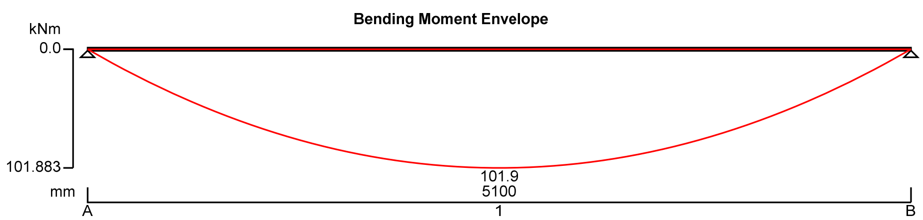
BEAM 1

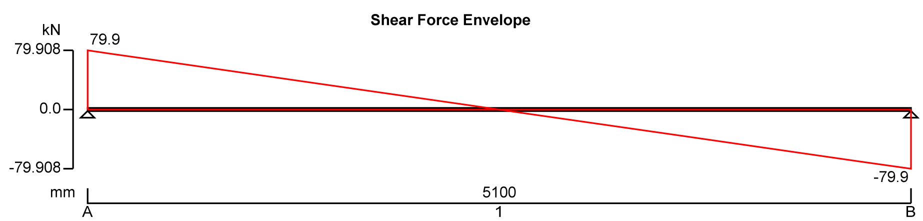
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Imposed full UDL 8.925 kN/m
	Dead full UDL 11.673 kN/m
	Dead self weight of beam × 1

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

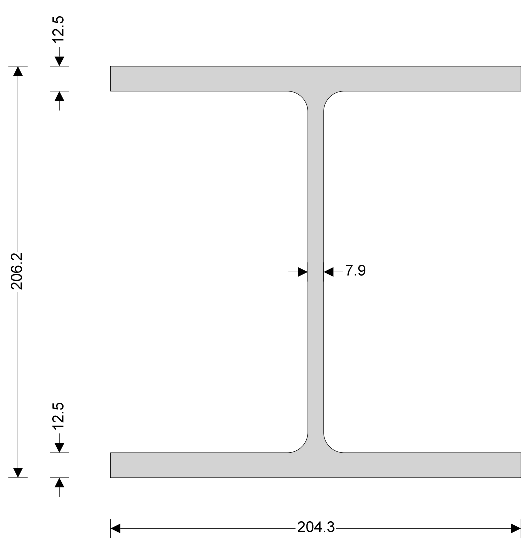
Maximum moment;	M_{max} = **101.9** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **79.9** kN;	V_{min} = **-79.9** kN
Deflection;	δ_{max} = **17.2** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **79.9** kN;	R_{A_min} = **79.9** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **31.1** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **22.8** kN	
Maximum reaction at support B;	R_{B_max} = **79.9** kN;	R_{B_min} = **79.9** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **31.1** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **22.8** kN	

Section details

Section type;	**UC 203x203x52 (BS4-1)**
Steel grade;	**S275**

From table 9: Design strength p_y

Thickness of element;	max(T, t) = **12.5** mm
Design strength;	p_y = **275** N/mm^2
Modulus of elasticity;	E = **205000** 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 × 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 = **160.8** mm

$d / t = 20.4 \times \varepsilon <= 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; b = B / 2 = **102.2** mm

$b / T = 8.2 \times \varepsilon <= 9 \times \varepsilon$; Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **79.9** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D$ = **1629** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v$ = **268.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **101.9** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **156** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = ((1.2 + 1.0) \times L_{s1} + 2 \times D) / 2$ = **5816** mm

Slenderness ratio; $\lambda = L_E / r_{yy}$ = **112.311**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; u = **0.848**

Torsional index; x = **15.838**

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.730**

Ratio - cl.4.3.6.9; β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **69.562**

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; α_{LT} = **7.0**

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.247**

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **418.1** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **398.2** N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **189.5** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; M_2 = **76.4** kNm

Moment at centre-line of segment; M_3 = **101.9** kNm

Moment at three quarter point of segment; M_4 = **76.4** kNm

Maximum moment in segment; M_{abs} = **101.9** kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \mathbf{101.9}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \mathbf{0.925}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = \mathbf{107.5}$ kNm

$M_b / m_{LT} = \mathbf{116.2}$ 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; $\delta_{lim} = L_{s1} / 250 = \mathbf{20.4}$ mm

Maximum deflection span 1; $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{17.248}$ mm

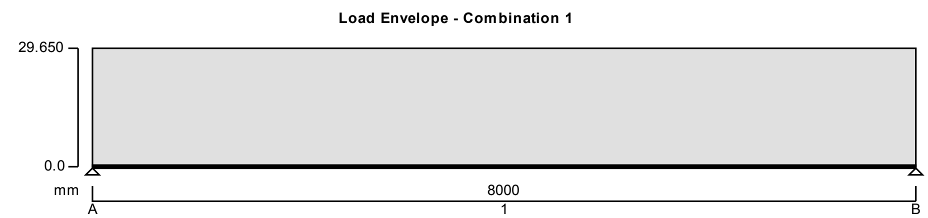
PASS - Maximum deflection does not exceed deflection limit

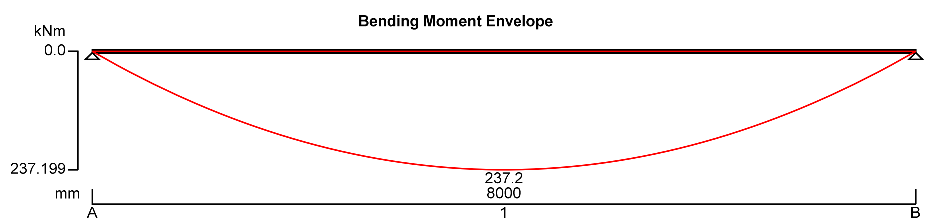
BEAM 2

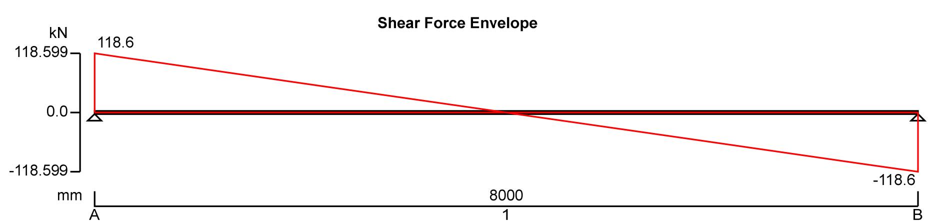
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Imposed full UDL 5.625 kN/m
	Dead full UDL 13.7 kN/m
	Dead self weight of beam × 1

Load combinations

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

Analysis results

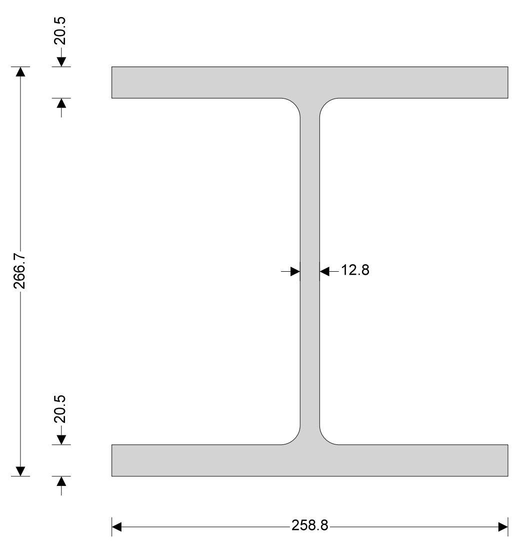
Maximum moment;	M_{max} = **237.2** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **118.6** kN;	V_{min} = **-118.6** kN
Deflection;	δ_{max} = **30.3** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **118.6** kN;	R_{A_min} = **118.6** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **59** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **22.5** kN	
Maximum reaction at support B;	R_{B_max} = **118.6** kN;	R_{B_min} = **118.6** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **59** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **22.5** kN	

Section details

Section type;	**UC 254x254x107 (BS4-1)**
Steel grade;	**S275**

From table 9: Design strength p_y

Thickness of element;	max(T, t) = **20.5** mm
Design strength;	p_y = **265** N/mm^2
Modulus of elasticity;	E = **205000** 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 × D

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **1.02**

Internal compression parts - Table 11

Depth of section; d = **200.3** mm

$d / t = 15.4 \times \varepsilon <= 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; b = B / 2 = **129.4** mm

$b / T = 6.2 \times \varepsilon <= 9 \times \varepsilon$; Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **118.6** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D$ = **3414** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v$ = **542.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **237.2** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **393.4** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = ((1.2 + 1.0) \times L_{s1} + 2 \times D) / 2$ = **9067** mm

Slenderness ratio; $\lambda = L_E / r_{yy}$ = **137.527**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; u = **0.848**

Torsional index; x = **12.393**

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.611**

Ratio - cl.4.3.6.9; β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **71.302**

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **34.951**

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

Bending strength - Section 4.3.6.5

Robertson constant; α_{LT} = **7.0**

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.254**

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **398** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **382.1** N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **180.7** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; M_2 = **177.9** kNm

Moment at centre-line of segment; M_3 = **237.2** kNm

Moment at three quarter point of segment; M_4 = **177.9** kNm

Maximum moment in segment; M_{abs} = **237.2** kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \mathbf{237.2}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \mathbf{0.925}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = \mathbf{268.3}$ kNm

$M_b / m_{LT} = \mathbf{290.1}$ 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; $\delta_{lim} = L_{s1} / 250 = \mathbf{32}$ mm

Maximum deflection span 1; $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{30.272}$ mm

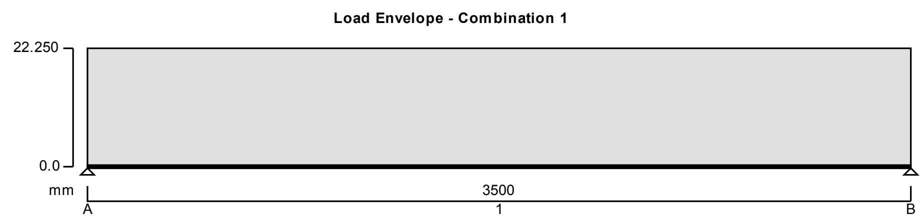
PASS - Maximum deflection does not exceed deflection limit

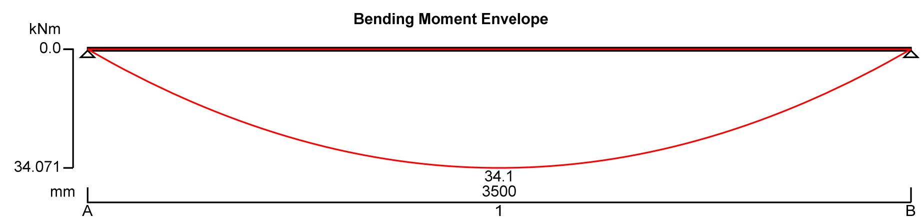
BEAM 3

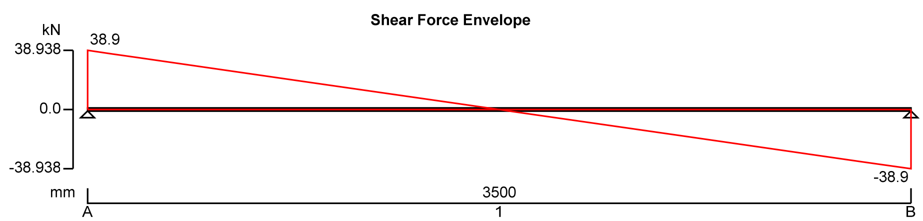
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Imposed full UDL 2.475 kN/m
	Dead full UDL 12.77 kN/m
	Dead self weight of beam × 1

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

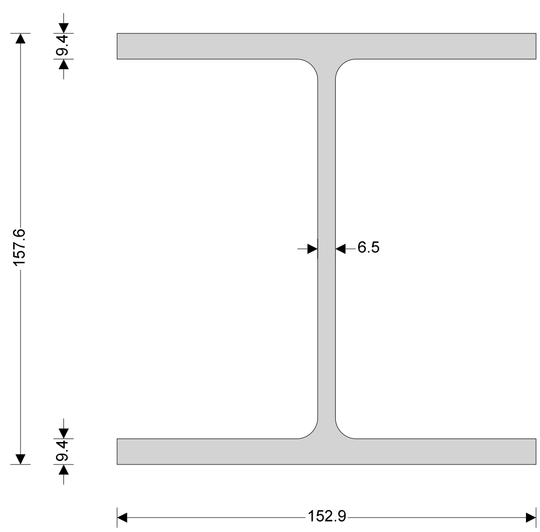
Maximum moment;	M_{max} = **34.1** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **38.9** kN;	V_{min} = **-38.9** kN
Deflection;	δ_{max} = **8.5** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **38.9** kN;	R_{A_min} = **38.9** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **22.9** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **4.3** kN	
Maximum reaction at support B;	R_{B_max} = **38.9** kN;	R_{B_min} = **38.9** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **22.9** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **4.3** kN	

Section details

Section type;	**UC 152x152x30 (BS4-1)**
Steel grade;	**S275**

From table 9: Design strength p_y

Thickness of element;	max(T, t) = **9.4** mm
Design strength;	p_y = **275** N/mm^2
Modulus of elasticity;	E = **205000** 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 × 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 = **123.6** mm

$d / t = 19.0 \times \varepsilon <= 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; b = B / 2 = **76.5** mm

$b / T = 8.1 \times \varepsilon <= 9 \times \varepsilon$; Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **38.9** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D$ = **1024** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v$ = **169** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **34.1** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **68.1** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = ((1.2 + 1.0) \times L_{s1} + 2 \times D) / 2$ = **4008** mm

Slenderness ratio; $\lambda = L_E / r_{yy}$ = **104.713**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; u = **0.849**

Torsional index; x = **15.999**

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

Ratio - cl.4.3.6.9; β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **66.737**

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; α_{LT} = **7.0**

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.227**

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **454.3** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **416.2** N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **196.4** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; M_2 = **25.6** kNm

Moment at centre-line of segment; M_3 = **34.1** kNm

Moment at three quarter point of segment; M_4 = **25.6** kNm

Maximum moment in segment; M_{abs} = **34.1** kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \mathbf{34.1}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \mathbf{0.925}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = \mathbf{48.7}$ kNm

$M_b / m_{LT} = \mathbf{52.6}$ 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; $\delta_{lim} = L_{s1} / 250 = \mathbf{14}$ mm

Maximum deflection span 1; $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{8.473}$ mm

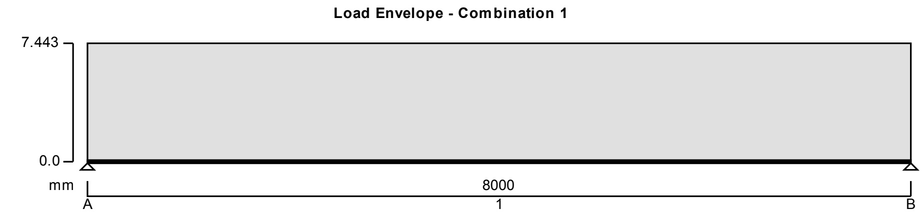
PASS - Maximum deflection does not exceed deflection limit

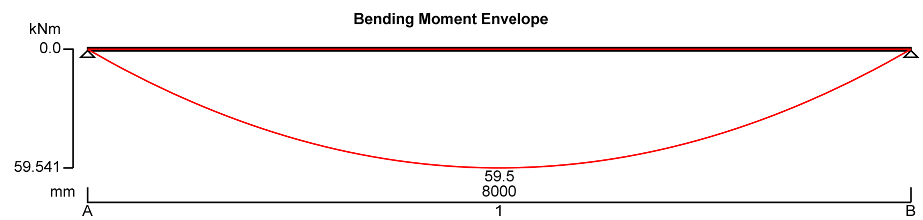
BEAM 4

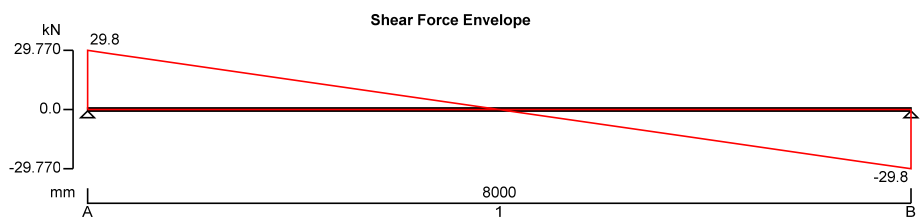
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05







Support conditions

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

Applied loading

Beam loads	Imposed full UDL 1.988 kN/m
	Dead full UDL 2.592 kN/m
	Dead self weight of beam $\times$ 1

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

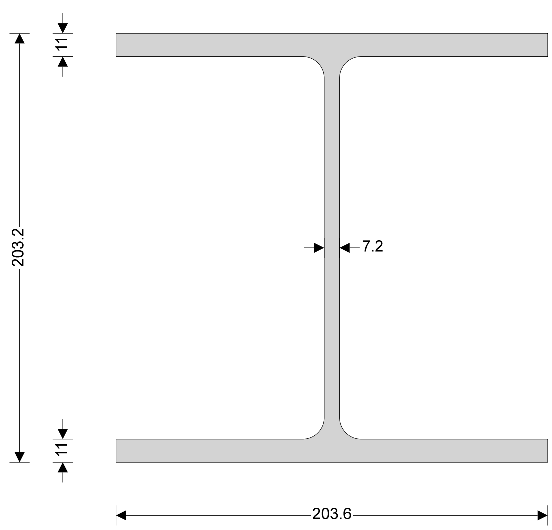
Maximum moment;	M_{max} = **59.5** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **29.8** kN;	V_{min} = **-29.8** kN
Deflection;	δ_{max} = **28.7** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **29.8** kN;	R_{A_min} = **29.8** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **12.2** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **8** kN	
Maximum reaction at support B;	R_{B_max} = **29.8** kN;	R_{B_min} = **29.8** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **12.2** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **8** kN	

Section details

Section type;	**UC 203x203x46 (BS4-1)**
Steel grade;	**S275**

From table 9: Design strength p_y

Thickness of element;	max(T, t) = **11.0** mm
Design strength;	p_y = **275** N/mm^2
Modulus of elasticity;	E = **205000** 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 × 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 = **160.8** mm	
	$d / t = 22.3 \times \varepsilon <= 80 \times \varepsilon$;	Class 1 plastic

Outstand flanges - Table 11

Width of section;	b = B / 2 = **101.8** mm	
	$b / T = 9.3 \times \varepsilon <= 10 \times \varepsilon$;	Class 2 compact

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force;	$F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **29.8** kN
	$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area;	$A_v = t \times D$ = **1463** mm^2
Design shear resistance;	$P_v = 0.6 \times p_y \times A_v$ = **241.4** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment;	$M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **59.5** kNm
Moment capacity low shear - cl.4.2.5.2;	$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **136.8** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling;	$L_E = ((1.2 + 1.0) \times L_{s1} + 2 \times D) / 2$ = **9003** mm
Slenderness ratio;	$\lambda = L_E / r_{yy}$ = **175.355**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter;	u = **0.847**
Torsional index;	x = **17.713**
Slenderness factor;	$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.642**
Ratio - cl.4.3.6.9;	β_W = **1.000**
Equivalent slenderness - cl.4.3.6.7;	$\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **95.242**
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;	α_{LT} = **7.0**
Perry factor;	$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.427**
Euler stress;	$p_E = \pi^2 \times E / \lambda_{LT}^2$ = **223** N/mm^2
	$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **296.6** N/mm^2
Bending strength - Annex B.2.1;	$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **133.4** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment;	M_2 = **44.7** kNm
Moment at centre-line of segment;	M_3 = **59.5** kNm
Moment at three quarter point of segment;	M_4 = **44.7** kNm
Maximum moment in segment;	M_{abs} = **59.5** kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \mathbf{59.5}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \mathbf{0.925}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = \mathbf{66.4}$ kNm

$M_b / m_{LT} = \mathbf{71.7}$ 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; $\delta_{lim} = L_{s1} / 250 = \mathbf{32}$ mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{28.661}$ mm

PASS - Maximum deflection does not exceed deflection limit

FLOOR JOIST

TIMBER JOIST DESIGN (BS5268-2:2002)

TEDDS calculation version 1.1.02

Joist details

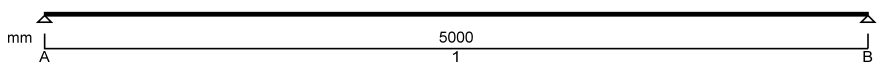
Joist breadth; b = **50** mm

Joist depth; h = **250** mm

Joist spacing; s = **400** mm

Timber strength class; **C24**

Service class of timber; **1**

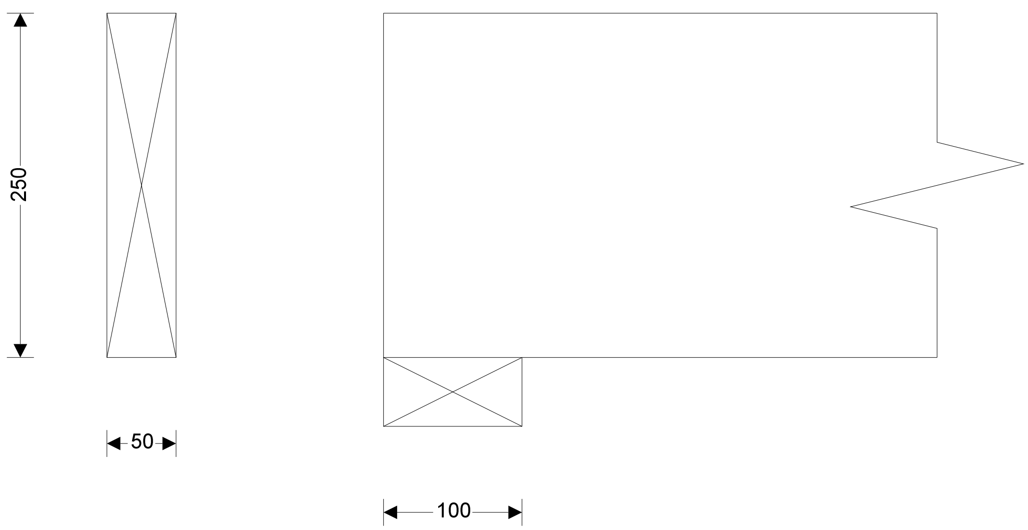


Span details

Number of spans; N_{span} = **1**

Length of bearing; L_b = **100** mm

Effective length of span; L_{s1} = **5000** mm



Section properties

Second moment of area; $I = b \times h^3 / 12 =$ **65104167** mm^4

Section modulus; $Z = b \times h^2 / 6 =$ **520833** mm^3

Loading details

Joist self weight;	$F_{swt} = b \times h \times \rho_{char} \times g_{acc}$ = **0.04** kN/m
Dead load;	F_{d_udl} = **0.75** kN/m^2
Imposed UDL(Long term);	F_{i_udl} = **1.50** kN/m^2
Imposed point load (Medium term);	F_{i_pt} = **1.40** 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 = **1.02**
Load sharing factor;	K_8 = **1.10**

<u>Consider long term loads</u>

Load duration factor;	K_3 = **1.00**
Maximum bending moment;	M = **2.947** kNm
Maximum shear force;	V = **2.357** kN
Maximum support reaction;	R = **2.357** kN
Maximum deflection;	δ = **11.332** mm

Check bending stress

Bending stress;	σ_m = **7.500** N/mm^2
Permissible bending stress;	$\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **8.417** N/mm^2
Applied bending stress;	$\sigma_{m\_max} = M / Z$ = **5.657** N/mm^2
	PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress;	τ = **0.710** N/mm^2
Permissible shear stress;	$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.781** N/mm^2
Applied shear stress;	$\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.283** N/mm^2
	PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane);	σ_{cp1} = **2.400** N/mm^2
Permissible bearing stress;	$\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **2.640** N/mm^2
Applied bearing stress;	$\sigma_{c\_max} = R / (b \times L_b)$ = **0.471** N/mm^2
	PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection;	$\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm})$ = **14.000** mm
Bending deflection (based on E_{mean});	$\delta_{bending}$ = **10.913** mm
Shear deflection;	δ_{shear} = **0.419** mm
Total deflection;	$\delta = \delta_{bending} + \delta_{shear}$ = **11.332** mm
	PASS - Actual deflection within permissible limits

<u>Consider medium term loads</u>

Load duration factor;	K_3 = **1.25**
Maximum bending moment;	M = **2.822** kNm
Maximum shear force;	V = **2.257** kN
Maximum support reaction;	R = **2.257** kN

Maximum deflection; δ = **9.555** mm

Check bending stress

Bending stress; σ_m = **7.500** N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **10.521** N/mm^2

Applied bending stress; σ_{m_max} = M / Z = **5.417** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.710** N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.976** N/mm^2

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.271** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane); σ_{cp1} = **2.400** N/mm^2

Permissible bearing stress; $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **3.300** N/mm^2

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b)$ = **0.451** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm})$ = **14.000** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **9.154** mm

Shear deflection; δ_{shear} = **0.401** mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **9.555** mm

PASS - Actual deflection within permissible limits

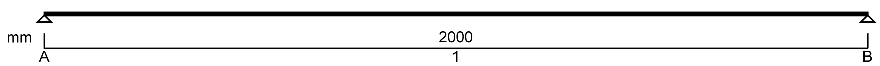
FLAT ROOF

TIMBER JOIST DESIGN (BS5268-2:2002)

TEDDS calculation version 1.1.02

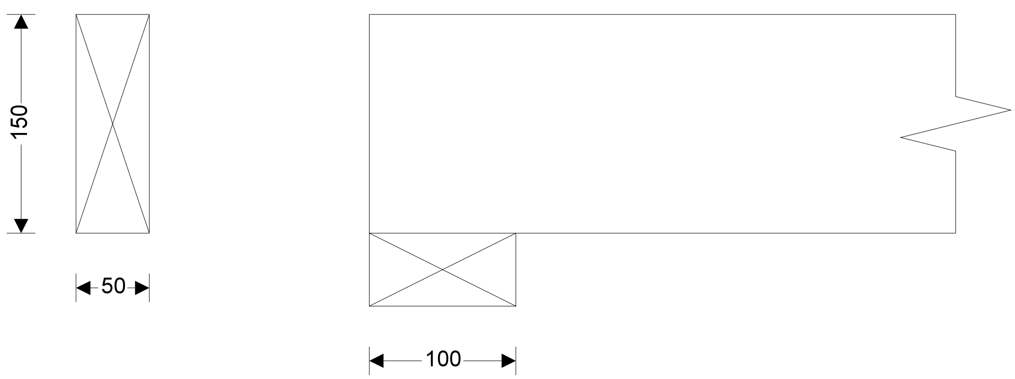
Joist details

Joist breadth;	b = **50** mm
Joist depth;	h = **150** mm
Joist spacing;	s = **400** mm
Timber strength class;	**C24**
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} = **2000** mm



Section properties

Second moment of area;	$I = b \times h^3 / 12 =$ **14062500** mm^4
Section modulus;	$Z = b \times h^2 / 6 =$ **187500** mm^3

Loading details

Joist self weight;	$F_{swt} = b \times h \times \rho_{char} \times g_{acc} =$ **0.03** kN/m
Dead load;	F_{d_udl} = **0.81** kN/m^2
Imposed UDL(Medium term);	F_{i_udl} = **1.50** kN/m^2
Imposed point load (Short term);	F_{i_pt} = **1.80** 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 = **1.08**
Load sharing factor;	K_8 = **1.10**

<u>Consider medium term loads</u>

Load duration factor;	K_3 = **1.25**
Maximum bending moment;	M = **0.475** kNm
Maximum shear force;	V = **0.950** kN
Maximum support reaction;	R = **0.950** kN
Maximum deflection;	δ = **1.415** mm

Check bending stress

Bending stress;	σ_m = **7.500** N/mm^2
Permissible bending stress;	$\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **11.130** N/mm^2
Applied bending stress;	σ_{m_max} = M / Z = **2.533** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress;	τ = **0.710** N/mm^2
Permissible shear stress;	$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.976** N/mm^2
Applied shear stress;	$\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.190** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane);	σ_{cp1} = **2.400** N/mm^2
Permissible bearing stress;	$\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **3.300** N/mm^2
Applied bearing stress;	$\sigma_{c\_max} = R / (b \times L_b)$ = **0.190** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection;	$\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm})$ = **6.000** mm
Bending deflection (based on E_{mean});	$\delta_{bending}$ = **1.303** mm
Shear deflection;	δ_{shear} = **0.113** mm
Total deflection;	$\delta = \delta_{bending} + \delta_{shear}$ = **1.415** mm

PASS - Actual deflection within permissible limits

<u>Consider short term loads</u>

Load duration factor;	K_3 = **1.50**
Maximum bending moment;	M = **1.075** kNm
Maximum shear force;	V = **2.150** kN
Maximum support reaction;	R = **2.150** kN
Maximum deflection;	δ = **2.710** mm

Check bending stress

Bending stress;	σ_m = **7.500** N/mm^2
Permissible bending stress;	$\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **13.355** N/mm^2

Applied bending stress; σ_{m_max} = M / Z = **5.733** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.710** N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **1.172** N/mm^2

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.430** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane); σ_{cp1} = **2.400** N/mm^2

Permissible bearing stress; $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **3.960** N/mm^2

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b)$ = **0.430** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003, 14\ mm)$ = **6.000** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **2.455** mm

Shear deflection; δ_{shear} = **0.255** mm

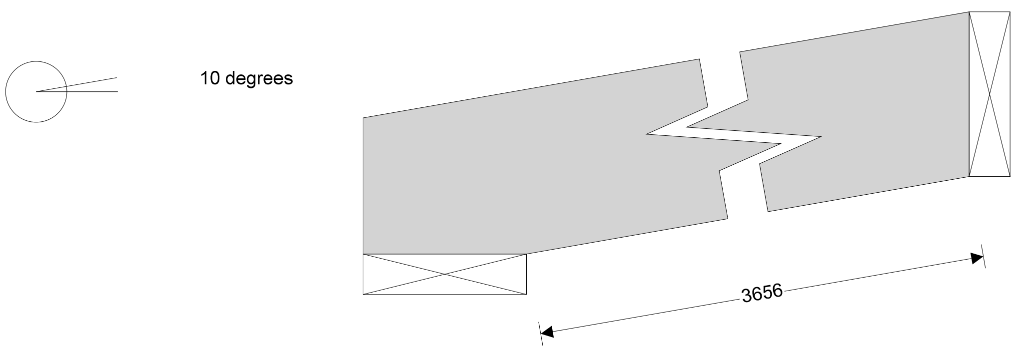
Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **2.710** mm

PASS - Actual deflection within permissible limits

RAFTER DESIGN

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections;	b = **50** mm
Depth of timber sections;	h = **200** mm
Rafter spacing;	s = **400** mm
Rafter slope;	α = **10.0** deg
Clear span of rafter on horizontal;	L_{clh} = **3600** mm
Clear span of rafter on slope;	$L_{cl} = L_{clh} / \cos(\alpha)$ = **3656** mm
Rafter span;	**Single span**
Timber strength class;	**C16**

Section properties

Cross sectional area of rafter;	$A = b \times h$ = **10000** mm^2
Section modulus;	$Z = b \times h^2 / 6$ = **333333** mm^3
Second moment of area;	$I = b \times h^3 / 12$ = **33333333** mm^4
Radius of gyration;	$r = \sqrt{(I / A)}$ = **57.7** mm

Loading details

Rafter self weight;	$F_j = b \times h \times \rho_{char} \times g_{acc}$ = **0.03** kN/m
Dead load on slope;	F_d = **0.75** kN/m^2
Imposed load on plan;	F_u = **0.75** kN/m^2
Imposed point load;	F_p = **0.90** kN

Modification factors

Section depth factor;	$K_7 = (300\ mm / h)^{0.11}$ = **1.05**
Load sharing factor;	K_8 = **1.10**

Consider long term load condition

Load duration factor;	K_3 = **1.00**
Total UDL perpendicular to rafter;	$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha)$ = **0.325** kN/m
Notional bearing length;	$L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)]$ = **5** mm
Effective span;	$L_{eff} = L_{cl} + L_b$ = **3660** mm

Check bending stress

Bending stress parallel to grain; σ_m = **5.300** N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **6.096** N/mm^2

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z)$ = **1.635** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain; σ_c = **6.800** N/mm^2

Minimum modulus of elasticity; E_{min} = **5800** N/mm^2

Compression member factor; K_{12} = **0.63**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **4.734** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A)$ = **0.369** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **14.241** N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.975**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.353**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; τ = **0.670** N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8$ = **0.737** N/mm^2

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A)$ = **0.089** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff}$ = **10.981** mm

Bending deflection; $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I)$ = **2.593** mm

Shear deflection; $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A)$ = **0.119** mm

Total deflection; $\delta_{max} = \delta_b + \delta_s$ = **2.712** mm

PASS - Total deflection within permissible limits

<u>Consider medium term load condition</u>

Load duration factor; K_3 = **1.25**

Total UDL perpendicular to rafter; $F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha)$ = **0.616** kN/m

Notional bearing length; $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)]$ = **9** mm

Effective span; $L_{eff} = L_{cl} + L_b$ = **3665** mm

Check bending stress

Bending stress parallel to grain; σ_m = **5.300** N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **7.620** N/mm^2

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z)$ = **3.104** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain; σ_c = **6.800** N/mm^2

Minimum modulus of elasticity; E_{min} = **5800** N/mm^2

Compression member factor; K_{12} = **0.59**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **5.562** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A)$ = **0.700** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **14.206** N/mm^2

Euler coefficient; $K_{eu} = 1 – (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.956**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.552**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; τ = **0.670** N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8$ = **0.921** N/mm^2

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A)$ = **0.169** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff}$ = **10.995** mm

Bending deflection; $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I)$ = **4.936** mm

Shear deflection; $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A)$ = **0.226** mm

Total deflection; $\delta_{max} = \delta_b + \delta_s$ = **5.161** mm

PASS - Total deflection within permissible limits

<u>Consider short term load condition</u>

Load duration factor; K_3 = **1.50**

Total UDL perpendicular to rafter; $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha)$ = **0.325** kN/m

Notional bearing length; $L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)]$ = **9** mm

Effective span; $L_{eff} = L_{cl} + L_b$ = **3664** mm

Check bending stress

Bending stress parallel to grain; σ_m = **5.300** N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **9.144** N/mm^2

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2/(8 \times Z) + F_p \times \cos(\alpha) \times L_{eff}/(4 \times Z)$ = **4.074** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain; σ_c = **6.800** N/mm^2

Minimum modulus of elasticity; E_{min} = **5800** N/mm^2

Compression member factor; K_{12} = **0.56**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **6.255** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha)+3 \times \tan(\alpha))/(2 \times A)+F_p \times \sin(\alpha)/A$ = **0.385** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **14.212** N/mm^2

Euler coefficient; $K_{eu} = 1 – (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.977**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.517**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; τ = **0.670** N/mm^2

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

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A)$ = **0.222** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff}$ = **10.992** mm

Bending deflection; $\delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha) / 48) / (E_{mean} \times I)$ = **5.700** mm

Shear deflection; $\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha)) / (5 \times E_{mean} \times A)$ = **0.296** mm

Total deflection; $\delta_{max} = \delta_b + \delta_s$ = **5.997** mm

PASS - Total deflection within permissible limits